Engineering Record

A Weekly Journal Devoted to Civil Engineering and Contracting

Published by McGraw Publishing Company, Inc.

Volume 69 NEW YORK, SATURDAY, FEBRUARY 7, 1914 Number 6

A $20,000 Engineer

AN engineer is now the highest-salaried public official in New York State. Announcement was made last week by the Public Service Commission that its chief engineer, Alfred Craven, would henceforth receive $20,000 yearly for his services in directing the work of building the city's new subway system, an increase of $5000 over the former salary. There is something more in this news than mere pecuniary gain for Mr. Craven. The commission's action is a recognition of the engineer's importance in public affairs, and as such will be glad tidings to those who have been trying to foster public sentiment to an appreciation of the real worth of the technically trained executive. It is significant that the new salary of the chief engineer of the Public Service Commission is double that of the Governor of the State of New York and $5000 greater than that of the Mayor of New York City.

Constitutional Change Needed

THE spending of a quarter of the business session of the annual meeting of the American Society of Civil Engineers week before last in debating a matter over which the meeting had no jurisdiction indicates the need of revising the section involved. The question arose over the proposed constitutional amendments. The situation was briefly outlined in the account of the meeting in the Engineering Record of Jan. 24. At the annual convention in Ottawa last June conflicting amendments were proposed. The meeting, therefore, availed itself of its privilege of referring the conflicting provisions to a committee, which, in accordance with the constitution, was to report to the annual meeting. The section of the constitution applicable reads, in part, as follows: . . . "which committee shall report to the next general meeting, whereupon the amendment shall be voted upon as hereinbefore provided." The method of voting referred to is by letter ballot. The discussion at the recent meeting was doubtless caused by the absence from this section of any provision for discussion and amendment of the committee's report, whereas amendments coming before the meeting directly as proposed are subject to discussion and amendment. After much argument the president ruled that the meeting had no jurisdiction—that discussion was out of order, that changes were not admissible, and that the amendments as they came from the committee would go to letter ballot. Naturally the question now comes: "If discussion and amendment and rejection of the committee's report are not admissible, why should the committee report to the general meeting?" If the clause is merely intended to set a date on which the committee is to report and the amendments to go to ballot, certainly the wording needs change. If report to the meeting was required merely that the committee might formally be discharged, provision could surely be made for that detail in some less confusing way. The constitution makers should direct their attention to the section involved, for if the machinery for constitutional changes is improperly designed difficulties are bound to recur.

Ohio Conservancy Act

WITH the general purpose of the Conservancy Act now before the Ohio Legislature all engineers are in sympathy. United effort is necessary if flood-protection measures are to be provided at reasonable cost for the smaller communities. There are some features of the measure, however, which in the judgment of this journal might well be improved. In the first place, the entire jurisdiction over the conservancy districts rests with the courts. Not only do they pass on the petitions for the establishment of such districts, but they exercise the only control over them provided by the law. The same arguments against court control can be advanced here as in the case of regulation of public-service corporations. The courts are already overworked and they have not the special experience and special machinery required for intelligent and effective control of such matters. Furthermore, though Sections 23 and 61 of the act do provide for co-operation among the districts themselves and any State or Federal agency, the co-operation is optional and no central organization is established for studying the possibilities and advantages of co-ordination. Were a "conservation commission," corresponding to the public service commission, given jurisdiction instead of the courts, it could not only exercise supervision more intelligently and effectively, but could also make studies to determine how districts should work together and realize the greatest returns from the expenditure. It is safe to predict that if the measure is passed in its present form, amendment will be necessary in the near future in order that State-wide plans may be made. Only such a plan of management will realize the greatest benefits for the State at large.

What Does Asphalt Maintenance Cost?

AN exhaustive investigation recently undertaken to ascertain costs of street maintenance going back twenty years or more developed the fact that there are very few cities in the United States that can tell what their asphalt street maintenance has cost or is costing. This condition has been recognized and talked about before, but the absolute dearth of this most essential information can only be realized when an attempt is made to obtain it. Buffalo is a notable and honorable exception to the rule that our cities have been and still are, to a great extent, ignorant of the amount they pay for maintaining their streets. It is true that some cities are improving in this respect, but the absence of old records makes comparisons impossible, and also renders it out of the question to show what the maintenance of a twenty-year-old pavement has cost per yard per year for the period of its life. Without these data it is difficult to see how any trustworthy conclusions can be drawn as to the relative merits of pavements, and while past omissions cannot now be corrected, it is time that every city, large or small, should begin to keep accurate maintenance data. Even where some record of this kind is kept it is often so incorrectly and carelessly compiled as to be valueless. One city, for example, lumps in its maintenance cost the expense of repairing plumbers' cuts or any other disturbance of the pavement. Such costs, of course, have nothing to do with maintenance properly speaking. In another city, in Pennsylvania, a lump sum is paid per year for maintenance of asphalt streets without any record whatever as to the actual cost of maintenance to the contractor, although it is known that the sum is in excess of what it should be. In some cases the date when a pavement was laid or resurfaced is unknown. To sum the matter up, it is almost the universal rule that our cities have no information upon which to base an expert opinion as to the durability and economy of their pavements, a condition so shocking on the face of it as to require no characterization, but to inspire a determination on the part of every well-managed city to start the compilation of accurate maintenance data without further delay.

A Reflection on the Profession

AT one of the highway sessions held in connection with the recent annual meeting of the American Society of Civil Engineers the subject of engineering organizations for highway work was discussed. Practically every speaker voiced the opinion that the adoption in New York State of this plan—which, of course, involved an engineer as head—would deal swift and horrible death to the venerable system of political highbindery there prevailing. That the mere adoption of a plan including an engineer as the responsible head is not a sure remedy for the evils that have beset New York's highway department is shown by the fact that even with two engineers on the board which guided the department's fortunes it was not possible to prevent a scandal equal to that caused by any muckraking-magazine campaign. In other words, character and backbone are needed as well as engineering ability. The profession as a whole has been badly hurt by the disclosures in New York, for it has been shown that a great number of engineers holding civil service appointments in the Barge Canal and highway departments have unwillingly contributed to the campaign funds of the controlling political party—unwillingly but without effective protest. This means that a body of men, strong in numbers, many of them the product of our technical schools, all claiming the temperament and training that fit one for engineering duties, have nevertheless meekly accepted a periodic plucking. No one could confuse the curt suggestion to contribute with a polite statement of the need of funds. These engineers were thoroughly aware of the fact that they were being held up. If one refused he knew that the weak places in his job-holding armor were exposed. The machinery might be set in motion to terminate his appointment; if the position were a higher one, an effort would be made—and usually with success—to break the stubborn one's determination by transfer without traveling expenses. It has been said that the concerted protest of twelve of these men could have broken up the system, and it seems plausible—but where were the twelve? Discussion of this subject is not pleasant. There is need for it, however, so that members of the profession may be steeled against any efforts to revive the system in New York, and may combine for its overthrow if it exists elsewhere. Their duty to themselves, even more than to the profession, requires that they stiffen their backbones and act.

New Method of Sewage Treatment

RECENT experiments at the Lawrence Experiment Station of the Massachusetts State Board of Health, as described in the article by H. W. Clark and G. O. Adams on page 158 of this issue, suggest that aëration of sewage for a period of some five hours in a 5-ft. tank containing vertical layers of slate about 1 in. apart may be utilized to greater advantage than hitherto thought possible. It appears that a substantial portion of the increase in stability, or reduction in the putrescibility, of the sewage is due to the clarification brought about by elimination of the semisoluble organic matters frequently spoken of as "colloidal" matter. This separation of non-settling substances contributing largely to the putrescibility of the sewage is promoted by the aëration and also by the growths or deposits which occur upon the surface of the slates. The tank is operated on the fill-and-draw plan, the sludge and detached growths from the slate surfaces quickly settling to the bottom of the tank after aëration ceases.

Direct oxidation of organic matter is not a controlling factor in removing organic matter, as indicated by numerous earlier investigations, and also by recent tests at Lawrence. It is important to note, however, that the treated sewage is claimed to have lost all of its offensive odor.

In some ways these important investigations give promise of more efficiency in bringing about the "de-solution" of objectionable non-subsiding organic substances than could be reasonably expected from studying any of the results of the experiments of Dr. Travis at Hampton, Norwich or elsewhere, or of Mr. Dibden at Devizes or elsewhere. Aëration seems to promote the separation of the colloids to a greater extent than anticipated by the English investigations or by Black and Phelps, who worked with tanks containing no slates or other colloiders.

The amount of air required seems to be comparatively small, both in point of quantity and expense. The experiments at Lawrence are striking in the elimination of odor from the sludge and the aid which the treatment is as a preliminary to filtration of the local sewage at rates far in excess of those hitherto employed in practice. It is stated that the purification data apply either to fresh or stale sewage.

A possible difficulty arises in that the aëration of sewage may of itself cause a serious nuisance from odors, such as reported in the aëration experiments of Dr. Dupre in London in the early '80s. This might be specially marked with stale sewages.

On the whole, the experiments are noteworthy in suggesting the beginnings of a new style of treatment, which, however, ought to be tried out on a larger working scale before final conclusions are drawn as to its usefulness for plants in practice.

As to Engineers' Fees

ONE of the most troublesome questions to a practising engineer is that of fixing the amount of the fees that he shall ask for his services. If he charges too much he loses work. If he charges too little he loses the respect of his client. The same man often errs in both directions at different times. In the serious endeavor to find a reasonable middle course he receives little aid from the engineering societies. Their membership is made up in large part of those who receive fixed salaries, who are contractors, or who are in other businesses, and this condition makes it impracticable for them to deal effectively with the question.

The engineer often envies the architect, who has a much more definitely fixed schedule of fees; but it seems hardly possible to use an equally definite schedule in engineering. This is particularly true of preliminary and special work not involving designs or the supervision of construction. Work of this character, although it does not pay the largest fees, is one of the most important from the standpoint of the development of natural resources.

The question is a most difficult one if the rate of compensation must be fixed in advance. This is because of the uncertainty as to the amount and kind of work that may be required. Good business men engaging engineers often want to make arrangements that fix definitely the amount of compensation that they may be called upon to pay. Such limits are often necessary, especially in public works and where work must be done under a limited appropriation. The engineer wants to meet his employer in this, and often goes farther than he ought.

The cases are common where an engineer, in order to meet competition, or, in the absence of sufficient knowledge as to what will be required, undertakes work for a fixed fee that proves to be inadequate. He then usually does as much as he thinks he can afford to do; sometimes he does much more; but it often happens that this is too little and the client is dissatisfied, not because he is paying too much but because the engineer is doing too little. In such a case an adequate fee would be equally advantageous to both parties. Cases of this kind are far too common.

If the work to be done is of a nature so definite that it is possible to know in advance just what is to be done, then the rate of compensation may be fixed with less likelihood of injustice. This is true of the work of making detailed plans and of superintending construction. In many other cases the nature of the work is not known and cannot be known in advance. The client knows that he is in trouble and needs help. He thinks that the engineer can help him. What the engineer can do remains to be determined by trial. It may turn out that the engineer is able to do, and actually does do for him, something that was not contemplated at the time he was engaged, and that it is much more useful than the work that was contemplated. Under such conditions the impracticability of fixing adequate compensation in advance is obvious.

The commonest way of meeting these uncertainties is by the use of per-diem rates. The plan of charging for professional services according to the time spent is accepted as an expedient. As such, it perhaps comes nearer to satisfying some of the impossible conditions of the case than any other feasible method. It fails to meet the requirement of the client for definiteness because no one can tell in advance how much time will be required. It also frequently works injustice. If an engineer, by reason of possession of special information and experience, is able to render services of substantial value to a client, the fact that only a little of his time is required should not operate to prevent his receiving a substantial fee. On the other hand, every engineer in active practice is sometimes connected with work which for one reason or another is less profitable than usual to his client, and in such cases, in a spirit of fair play, the engineer's usual per-diem rate may represent more than a fair fee.

It is far easier to determine what a fair compensation is after the services are rendered. Then all the essential elements are known with more or less definiteness. Some of the most important engineering work is performed without other arrangement between engineer and client than the understanding that there is to be fair play and that the charges will be reasonable. Under this plan an engineer can often render valuable services to his client which would not be possible if he were restricted in what he may charge by an agreement made before the conditions were known. When the client and engineer can trust each other fully this method has great advantages. Under it there is better chance of the fullest service and the greatest advantage to both parties. In other professions the advantages of this method, or perhaps the difficulty of other methods, is so great that it has become customary, and attempts to fix compensations in advance are unusual. No one expects a doctor to name beforehand the fee that he is to receive when he is called to see a patient.

One reason why engineering work is not more frequently performed in this way is because clients and engineers have not sufficient confidence in each other. The reason for this may be found in the great diversity in the kinds of engineering work, in the fact that work of a particular kind is often widely scattered, and that the engineer and client seldom know each other as well as men are apt to know their lawyers and their doctors.

Whatever the cause, this lack of confidence is unfortunate and tends to limit the field of usefulness of engineers. This is one of the matters that must be worked out and corrected. Doing it will add to the dignity and usefulness of the profession. The engineer must be faithful to his client's interest all the way through, up to and including the amount of his charge, and must deserve the confidence and trust of his client. On the other hand, if clients can be induced to take the pains to select engineers whom they can fully trust, it will go a long way toward bringing about better conditions.

Panama Canal Influence on the Pacific Northwest

THE influence which the Panama Canal is expected to have on Pacific ports has incited much optimism in the West, and has not failed to draw comment from the more conservative element. There is no doubt that it will be some time before the maximum benefits of the canal will accrue, and care should be taken not to create too great hopes for first results lest depression follow as a natural reaction. On the other hand, it is pleasing to note that capital has not been lacking for new harbor and dock work, which betokens widespread confidence as to the ultimate effect of the canal on Pacific Coast traffic. Each port from Prince Rupert to San Diego has undertaken improvements, and the extension of railway lines to the Coast, together with the terminal facilities involved, has brought the total development cost encouraged by the canal up to a figure considerably larger than the total amount expended on the canal itself. In fact, an impartial observer visiting the Coast in the interests of the London "Times" places the cost of Pacific Coast port work undertaken in preparation for the opening of the canal at about $500,000,000.

It is worth while to consider the extent of the improvements at the various ports. Aside from the harbor problems, the effect which these developments have on tributary resources is a matter of much interest. From the Pacific Northwest, particularly, there come favorable reports of the extent to which the country is being opened up by the developments under way.

At the mouth of the Columbia River, where the improvement of the channel has proceeded with a new impetus since the Panama Canal was assured, the amount expended on jetty work alone is well over $10,000,000. Army engineers recently announced that on Feb. 2 the ten months' work planned for 1914 would be started. When this is well under way 4000 tons of rock are to be dumped daily at the jetty site. It is hoped that after this season's work there will be, at least, a satisfactory channel, with a minimum depth of 30 ft.

The Columbia River has a total watershed area of 254,388 sq. miles, and Government statistics show that of the portion arable, only about 18 per cent is under cultivation. However, the total wheat production in the three Northwestern States last year is given as 88,000,000 bushels. In the Columbia River basin the freight rate on wheat is 0.75 cent per ton-mile, but this may be affected by the Dalles-Celilo canal, now nearing completion, which will open to navigation an additional 200-mile stretch on the upper Columbia. It is interesting to note that the wheat rate between Chicago and New York, where there is a competitive water route, is only 0.147 cent per ton-mile.

In the three Northwestern States most affected by navigation on the Columbia River are said to be 300 billion feet of commercial timber. Oregon timbermen estimate that when vessels with a 25 to 28-ft. draft can cross the Columbia River bar, this lumber can be supplied, via the canal route, to New York and Philadelphia markets cheaper than yellow pine can be brought from the Southern States.

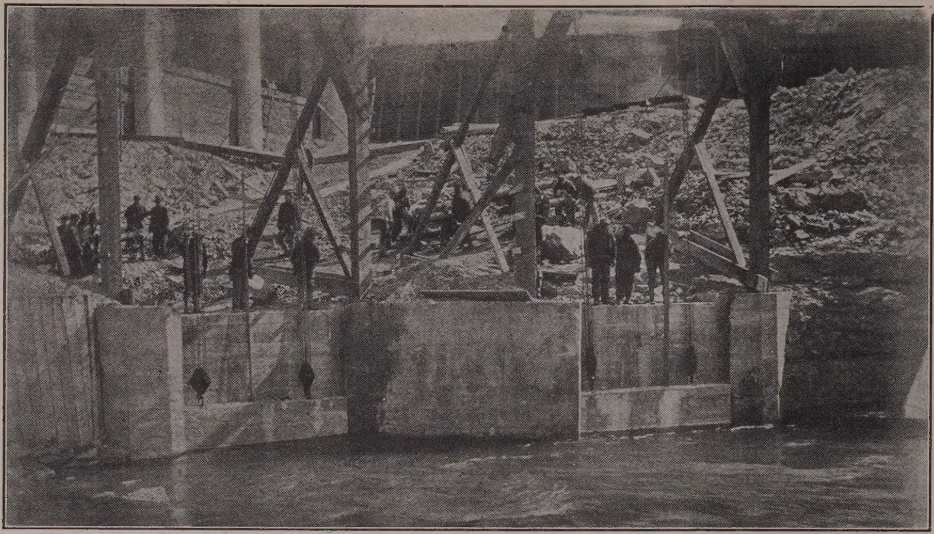
Tacoma contemplates a 6-mile canal system which will develop a large tide-flat area, and a $4,000,000 municipal expenditure for port improvements is to be supplemented by a half million of Government funds to be used in channel dredging. These developments are to afford 42 miles of dock frontage. Another $1,000,000 will be spent, altogether, for dock construction by the Puget Sound cities of Olympia, Everett and Bellingham. Portland and Tacoma have shipped wheat to England and France to some extent—12,000,000 bushels in 1911. The water route from Portland to Liverpool will be reduced 6000 miles by the canal, which is expected to afford freight rates so much lower than formerly that new markets in Europe will open to products from this section.

Seattle is now completing within its boundaries a $5,000,000 canal, which connects tidewater with Lake Washington and adds 37 sq. miles of navigable fresh water to the harbor area. The 10 miles of dockage now in use are being improved and extended by the city at a cost of about $8,000,000, while $3,000,000 more is being used to adapt the Duwamish River to the needs of manufacturing industries. Unlike the Columbia River, Puget Sound has always been open to vessels of deep draft, which can usually proceed directly to their berths without special pilot; but this is offset by the fact that Sound ports are cut off from the inland areas by a mountain barrier which requires a heavy-grade rail haul from which the Columbia River ports are free.

The Canadian Government has undertaken extensive port improvements at Victoria, the provincial capital, which include breakwater, pier and drydock construction, and for which the preliminary estimates total more than $10,000,000. Vancouver is fortunately located as a port in that the Fraser River affords easy railroad grades inland, while Burrard Inlet, an arm of the sea, gives unobstructed harbor facilities. This port, like Seattle, is about 160 miles by channel from the point where the Straits of Juan de Fuca open to the Pacific. The Dominion Government has under way at Vancouver channel and dike work to cost more than $4,000,000, while a floating drydock and shipbuilding berths are projected. Private companies have secured extensive foreshore rights and claim to be preparing for development on a stupendous scale, but actual construction has not been undertaken.

Grain production in three of the western provinces of Canada has increased during the past decade at the rate of 100 per cent in five years, and the railway companies admit that it has been impossible to extend their lines fast enough to keep abreast of the development. There is no doubt that the Panama Canal has been a factor in spurring construction work on the western ends of the new transcontinental lines (even though the trade of the Orient has played a large part in the development), nor is the haste to be wondered at under the circumstances. The distance from Edmonton to Fort William, on Lake Superior, is 1260 miles, while Edmonton grain can reach tidewater 735 miles away at Vancouver, or 825 miles over the Grand Trunk Pacific, which will have a maximum westbound grade of 0.5 of 1 per cent. Besides being an all-winter port, Vancouver's location should make it possible for one boxcar to take out twice as much grain in a season by cutting the length of haul in half. But aside from considerations of this nature, any new grain outlet would be eagerly sought because conditions have reached a point where the time element enters and the crop has to be rushed to tidewater. In recent seasons millions of bushels of Canadian wheat had to be shipped annually by rail from Buffalo to New York—a significant factor in the rush construction of the extensions through British Columbia.

There will soon be opportunity for most interesting comparative studies of the trade increase at the various Pacific Coast ports and the extent to which preparatory development work has paved the way for it. Perhaps the most interesting situation, and the one, no doubt, which will take shape first, centers at Vancouver. The two new Canadian railroads to be completed next year should not only affect freight rates on territory already served, but should open up vast areas which hitherto have had no influence on commerce.



Lowering Reinforced Concrete Gates Which Close Unwatering Tunnels

Horseshoe Falls Hydroelectric Plant

Canadian Station of 19,500 Horsepower, with Two 12-Foot and Two 9-Foot Penstocks, Operating under Head of 70 Feet

By HAROLD S. JOHNSTON
Resident Engineer, Calgary, Alberta

CALGARY, situated in the foothills on the eastern slope of the Rocky Mountains at the junction of the Bow and Elbow Rivers, has long felt the need of a supply of cheap power for general municipal purposes and as an inducement to manufacturers to locate within the city. The city's population is 60,000. Many years ago a small hydroelectric plant, operating under a head of only 8 feet, was established within the city limits. As the load upon this plant exceeded its capacity, steam units were added, sawmill refuse being largely used as fuel. Later the city established for general lighting and street-railway purposes a steam plant, which, starting with small units, gradually increased in capacity, until in 1911 a station of about 2000 kw was in use. This plant, extended in capacity from time to time, could not be operated economically, and since 1907 the city has been negotiating with different parties for a supply of power ample for all needs of the municipality and at a reasonable price.

Selecting Site

The Bow River contains the best power sites within transmission distance of Calgary. Above the site selected for the development the river has a drainage area of about 2000 square miles. The Horseshoe Falls site is about 50 miles west of Calgary and ¾ mile from the main line of the Canadian Pacific Railway. This site was secured in 1907 by the Calgary Power & Transmission Company, which is largely composed of citizens of Calgary, and plans and specifications were prepared for a hydroelectric development. These plans, however, were never carried out, and in 1908 the Calgary Power Company of Montreal purchased the assets of the former company, entered into a contract with the city for a supply of power, and retained Smith, Kerry & Chace, of Toronto, Ont., as engineers to design and superintend the construction of the plant.

Geological Formation

The plant is at a bend in the river where there are two falls, the upper having a drop of 8 ft. and the lower a drop of 14 ft. within the comparatively short horizontal distance of 450 ft. The crest of each fall is a hard layer of quartzose sandstone about 5 ft. thick. Between these two layers is a vertical distance of about 70 ft., made up of bands of softer shales and sandstones. Just below the upper fall a pool, 60 to 70 ft. deep, had been eroded by the scouring action of the falling water. When the location of the works was studied it was impossible to tell exactly how far the hard top layer, on which the dam would be founded, had been undercut by the action of the water. It was, therefore, determined to keep the structures as far upstream as the conditions permitted and to build at right angles to the thread of the stream rather than in the form of a horizontal arch around the Horseshoe.

The advantage of the horizontal arch type would have been a foundation throughout on the hard top layer of sandstone, whereas the dam, as originally designed, had its western section, designed as a retaining wall, founded upon the hard black shale overlying the 5-ft. layer of quartzose sandstone.

The sluiceway and spillway openings are located at the east end of the dam, which, as before stated, is founded upon the hard sandstone. The portion next the west end was originally designed as a retaining wall for which the black shale foundation was considered to be safe. Throughout the shale, between the hard sandstone layers, are narrow dikes of exceedingly hard and brittle rock.

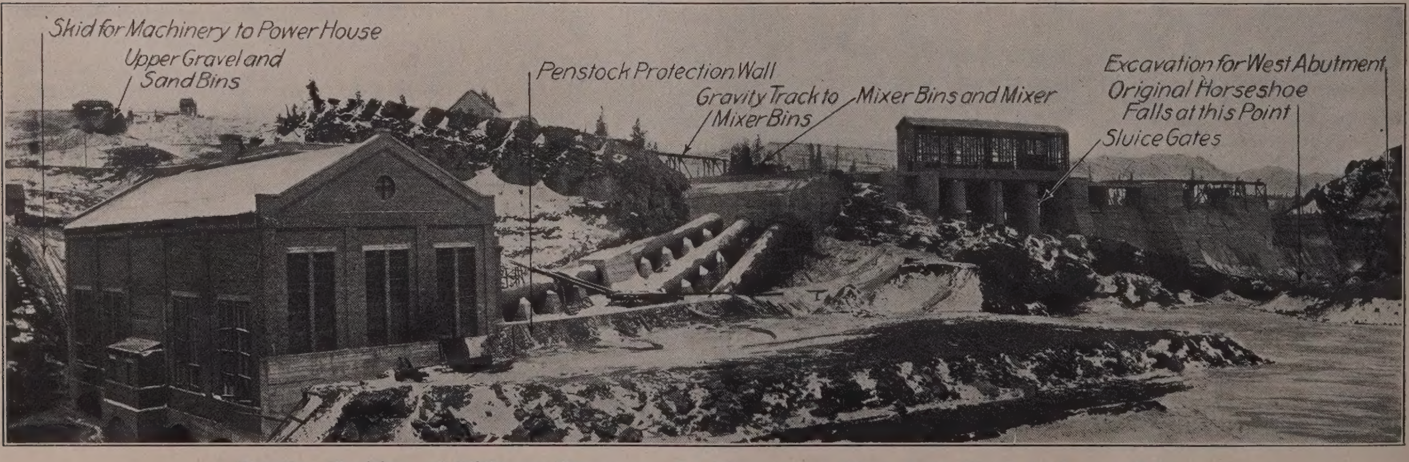
River Control

As soon as the general position of the dam was decided upon contracts were let for the driving of two unwatering tunnels for the cofferdam, and for the excavation for the headworks and railway sidings. Each unwatering tunnel is 12 ft. wide and 8 ft. in clear height. One of the tunnels is 147 ft. long and the other 150 ft. Both were graded to a slope of 5 ft. in their length. The interior was left very rough, no particular effort being made to trim the walks. The discharge of the two tunnels, with a head of 15 ft. over the intake, was about 3500 sec. ft.

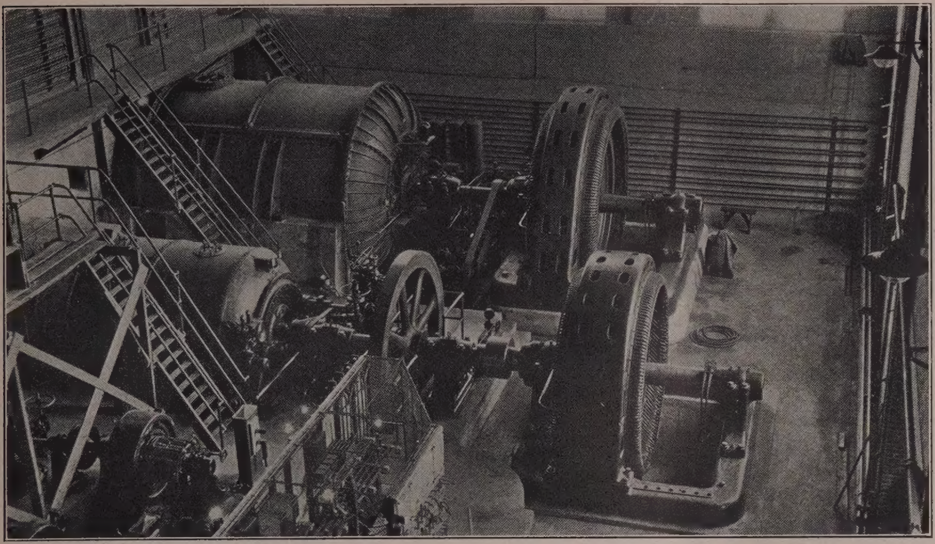
When the unwatering tunnels were completed and small concrete headworks with stoplog guides built, the cofferdam for the dam was constructed and the river diverted. This cofferdam was of the stone-filled crib type with sheeting on the upstream face to cut off the greater portion of the leakage, and it was rendered watertight by double sheeting on the downstream face. A concrete toe between the bottom of the sheeting and the rock made a watertight joint.

Headworks

The headworks deck is about 100 x 40 ft., and the trash racks on the face are formed of bars 4 x ⅜ in. spaced 2 in. apart in the


Skid for Machinery to Power House
Upper Gravel and Sand Bins
Penstock Protection Wall
Gravity Track to Mixer Bins
Mixer Bins and Mixer
Excavation for West Abutment
Original Horseshoe Falls at this Point
Sluice Gates

Relative Position of Different Structures, Looking across River from Brink of Lower Falls

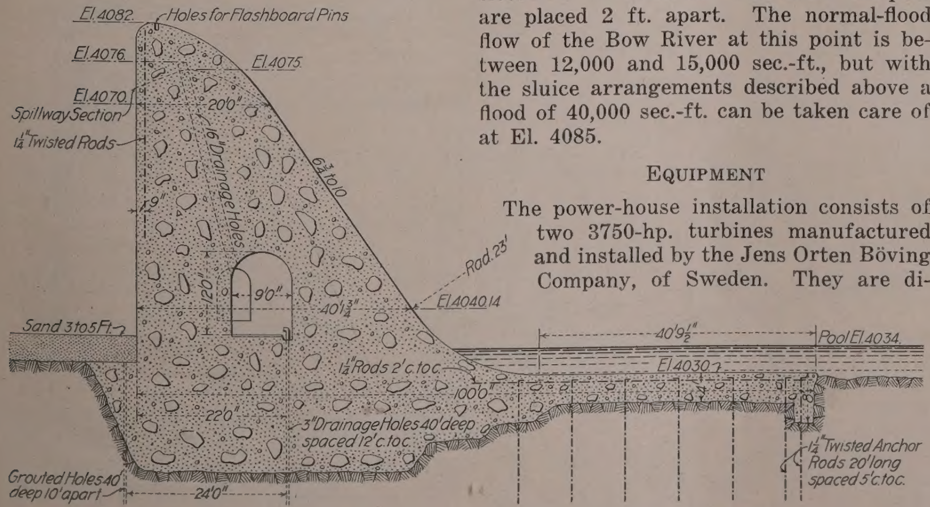


Arrangement of Turbines and Generators in Interior of Power House

clear. The upper 14 ft. of the racks are made up of oak bars. The intakes to the penstocks are at an angle of 60 deg. with the face line of the headworks. Near the back thimbles are set in the concrete for connection to the steel pipes. The intakes can be closed by means of butterfly valves with square tongues, manufactured by Glenfield & Kennedy, of Kilmarnock, Scotland. For additional protection stoplogs are provided. Just back of the butterfly valves 2 x 2 ft. air vents lead from the penstock tubes. They are also provided with rungs and serve as manholes, giving access to the penstocks.

There are four steel penstocks approximately 250 ft. long, the two outside ones being 12 ft. in diameter and the two inner ones 9½ ft. The latter are of medium steel ⅜ in. thick throughout, the former of ½ in. All are supported on concrete saddles, spaced about 15 ft. apart, which surround them up to their centers. Each penstock is anchored in the center of its length by an anchor block of ample proportions. Each pipe is also provided with two expansion joints, one at the upper end and one adjacent to the thrust block at the power-house wall. These expansion joints are of the slip-joint type, packed with hemp boiled in lard.

An ice and trash chute is provided in line with the headworks racks and at the angle point between the main dam and the headworks. The headworks are covered with a gate house made of steel and plaster and cantilevered over the racks 10 ft. for protection from ice during the winter. Swinging timber doors below the building extending down to the water surface serve to keep cold and wind out. They are hinged at the point of support so as to take up any ice thrust without danger to the building.



Cross-Section of Dam and Location of Grouted Holes and Weepers

SLUICE GATES

Four steel sluice gates are provided with operating mechanism inside the gate house. These gates are about 14 ft. wide by 23 ft. high, and are operated by oil under a pressure of 250 lb. per square inch. All parts are tested for a pressure of 800 lbs. per square inch. Large inspection tubes are placed in the piers between the gates and in line with them and contain the cast-iron cylinders for the rams connected directly at the upper end to the gates. The oil-pressure inlets are at the top and bottom of the cylinders. The gates have large bearing wheels, bronze bushed, running on steel tracks placed on columns. A 35-hp., 550-volt alternating-current motor is used to drive a triplex pump for furnishing the oil pressure.

In addition to the sluice gates, four stoplog sluiceways are provided, with their sills at El. 4075, and the normal operating level is at El. 4082. There are 140 ft. of spillway section at El. 4082. On this 2 ft. of flashboards have been placed, consisting of 2-in. planks tied to 3-in. oak pins set in pipe sleeves in the concrete. The above oak pins are placed 2 ft. apart. The normal-flood flow of the Bow River at this point is between 12,000 and 15,000 sec.-ft., but with the sluice arrangements described above a flood of 40,000 sec.-ft. can be taken care of at El. 4085.

EQUIPMENT

The power-house installation consists of two 3750-hp. turbines manufactured and installed by the Jens Orten Böving Company, of Sweden. They are directly connected to two horizontal shaft, 2500-kva., three-phase, 60-cycle, 12,000-volt, 300-r.p.m. generators. Two turbines of 6000-hp. capacity, manufactured by the Wellman-Seaver-Morgan Company, of Cleveland, Ohio, are directly connected to two horizontal-shaft generators of 4000-kva capacity, three-phase, 60-cycle, 12,000-volt, 225 r.p.m. There are also two 330-hp turbines directly connected to two horizontal shaft, 175-kw, 125-volt, 700-r.p.m. exciters. The generators and exciters were furnished and installed by the Canadian General Electric Company.

Current is carried from the generators to either of two sets of busbars, one set supplying the two 12,000-volt outgoing lines to the Exshaw cement plant and the other set supplying the transformers which step up the voltage to 55,000 volts for the transmission to Calgary.

TRANSMISSION LINES

The transmission lines are of 2/0 aluminum cable supported on pin-type insulators on wooden poles. The Exshaw line is a double-circuit line, 8 miles long. There are two lines to Calgary paralleling each other for 11 miles, then branching so that they are about 16 miles apart at the most distant point, joining again about 4 miles from the terminal station and following by suburban streets to the station in the most easterly part of Calgary. After the first 13 miles both lines run on opened or unopened roads. Both lines are on wooden poles, with an overhead ground wire on each line, but with a different system of construction. A private telephone circuit is carried on the same poles as the power wires.

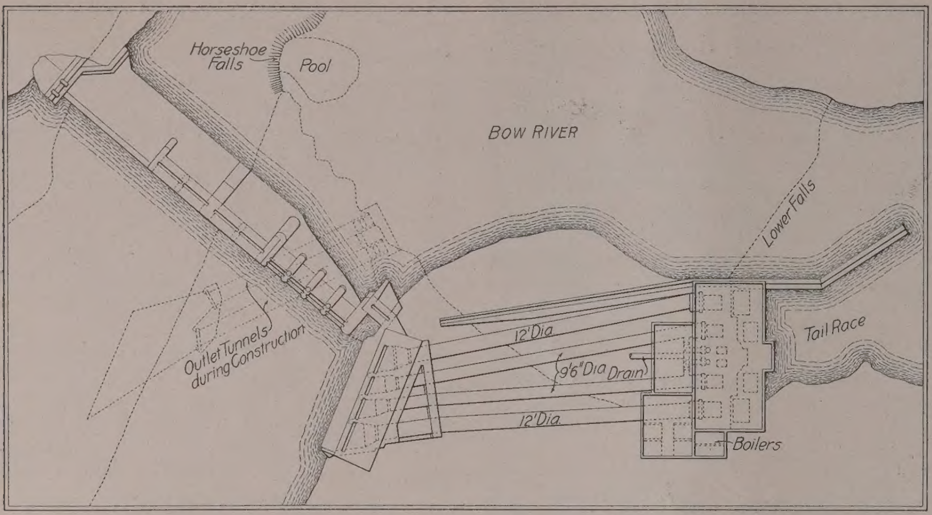
The transforming station at the power house contains at present four 3000-kva, oil insulated, water-cooled, three-phase transformers. Space is provided for one spare transformer.

The substation at Exshaw contains four 700-kva oil-insulated, water-cooled, three-phase transformers stepping down the voltage from 12,000 to 600 volts to supply a cement mill close at hand.

The terminal station in Calgary steps down the voltage from 55,000 to 12,000, 2400 and 600 volts for distribution to Calgary and the Canada Cement Company's plant adjacent to the station. There are at present installed in this station two 3000-kva and two 1250-kva, oil-insulated, water-cooled, three-phase transformers, with the necessary switching equipment for supply at the three voltages. Space is provided in the building for two additional 3000-kva transformers and additional switching equipment. It is a reinforced concrete building, steam heated. The water for the cooling apparatus and the boilers is taken from a reservoir tank belonging to the Canada Cement Company, but an auxiliary connection is also made to the city water main.

The switching equipment at the power house is remote-controlled, electrically operated from a control desk located on a gallery overlooking the generator room. Westinghouse type C oil switches mounted in concrete compartments are installed on the 12,000-volt circuits, while Westinghouse type G. A. oil switches are provided for the 55,000-volt transmission line. Two Tirrill regulators with line-drop compensators are provided to regulate the voltage at Calgary and Exshaw.

The switching equipment at the terminal



General Plan of Development, Giving Location of Diversion Tunnels

station is also remote-controlled, electrically operated, from a control board. Westinghouse type E oil switches in concrete apartments are installed on the 600, 2400 and 12,000-volt circuits. Electrolytic lightning arresters and air-type choke coils are in use on the transmission lines in all the stations.

POWER HOUSE

The power house and switch house have concrete foundations and the walls are concrete for 14 ft. and brick for the remainder. The transformer house is a brick building throughout, on concrete foundations. The switch house and transformer house have burlap, pitch and gravel roofs, while the power house has a gable roof covered with metal shingles. All the buildings are heated with steam from a boiler house placed outside the power house at one end. The power house itself is 118 ft. long by 56 ft. wide. A 35 ton Northern Engineering Works crane spans the whole width of the building.

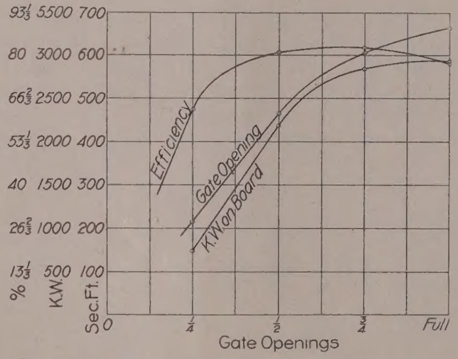
A test was made in October, 1911, on one of the 3750-hp Swedish turbines, a representative of the manufacturing firm being present. The water flow to the turbine was measured by a Pitot tube in the penstock about 40 ft. from the wheel. A water rheostat was built and placed in the tailrace to which the current at 12,000 volts was carried. The results of this test are shown on the accompanying curves.

GRAVITY SECTION DAM

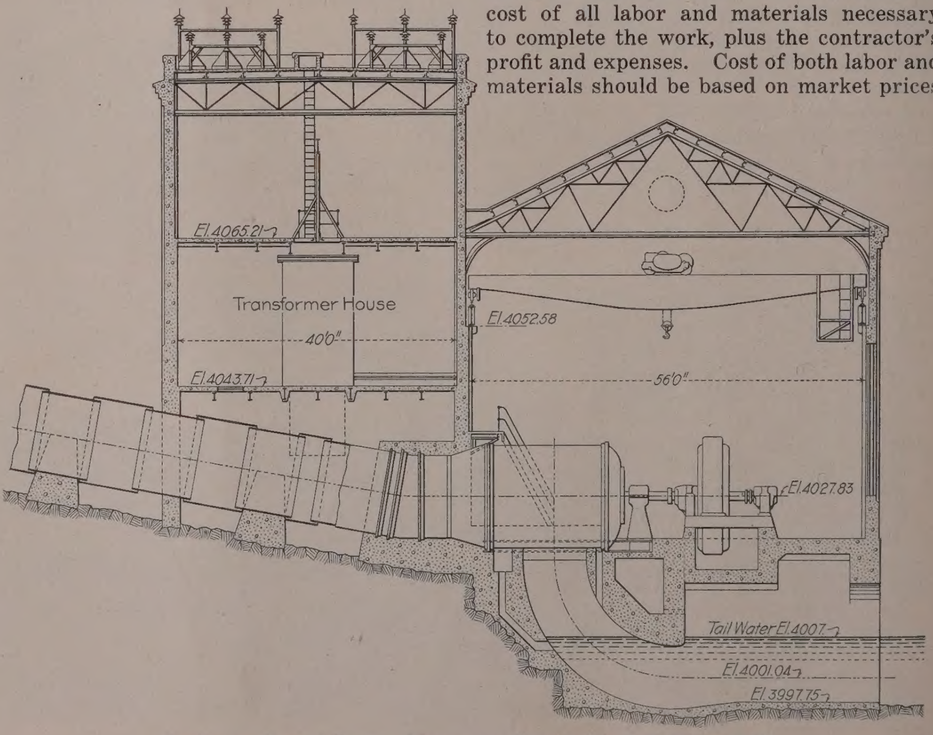
During 1912 a storage amounting to 44,000 acre-ft. was obtained at Lake Minnewanka, about 30 miles from the plant, by means of a gravity-section concrete dam only 40 ft. long at the base and 80 ft. at the crest across Devil's Canyon. Property rights and the fact that the storage was in one of the National Parks of Canada precluded the possibility of raising the lake more than 12 ft. Economically and physically, a much larger storage might have been obtained. Although the dam was built at a point 5 miles from the railway, and construction was started in weather which reached a temperature of 15 deg. below zero, work was finished three months from the time the construction force began operations.

The plant has been in operation since May, 1911. The Calgary Power Company has now under construction at Kananaskis Falls, 2 miles further up the river, another plant with approximately the same head—70 ft.

The Walker Fyshe Company, of Montreal, was the general contractor for the work, and Smith, Kerry & Chace, of Toronto, were the designing and superintending engineers until May, 1911, when the Calgary Power Company finished the construction work with its own complement of men.



Test Curves for Main Unit



Cross-Section of Power House on Center Line of Main Unit

Proposed Schedule of Fees

Abstract of Preliminary Report Presented by Committee on Code and Ethics to Connecticut Society of Civil Engineers

A SCHEDULE of charges for professional services is submitted to the Connecticut Society of Civil Engineers in the preliminary report of the committee on code and ethics. The committee, consisting of Edward W. Bush, John C. Tracy and Herbert C. Keith, believes that, in ordinary practice, a fair and proper compensation for engineering services will lie somewhere between the minimum and maximum charges given in the schedule. It recognizes, however, that the magnitude of the work and the experience or reputation of the engineer frequently justify a decrease or increase from the schedule, which is as follows:

THREE METHODS

Charges may be based on any of the following methods, or a combination of the same—A, a percentage of the cost of the work; B, a fixed sum, or C, a per diem rate.

A—Percentage of Cost of Work.—(1) For preliminary conferences, preliminary studies, estimates and reports, fees shall be from 1 to 1½ per cent of cost of work; (2) for the services outlined in (1), for the design, and for the preparation of contract drawings, specifications, and contracts, there should be a total charge of from 2½ to 5 per cent of cost of work; (3) for the services outlined in (2), for advice in letting contracts, and for general supervision during construction, a total charge of from 4 to 8 per cent, and (4), for full professional services, including complete supervision and inspection of construction, a total charge of from 6 to 12 per cent.

When no contracts are let, but the work is done under the business management of the engineer, the additional compensation depends to such an extent upon conditions that it should be a matter of special agreement between the engineer and his client.

In the preliminary stages cost of work is based on estimated cost; later it is based on cost of all labor and materials necessary to complete the work, plus the contractor's profit and expenses. Cost of both labor and materials should be based on market prices current when the work was ordered, labor to be fully paid and materials to be considered as new.

Payments should be made from time to time as services are rendered, the amounts to be based upon the proportion of services completed.

B—A Fixed Sum.—(1) A fixed sum for services may be agreed upon in lieu of a percentage or a per diem charge, and an additional charge made for expenses; (2), there may be a fixed total sum for all services of principal or assistants and for all attending expenses.

A fixed sum may be charged for a portion or all of the items of preliminary surveys, studies, examinations, reports, plans, specifications and supervision, etc., either with or without attending expenses.

C—Per Diem Rate.—(1) For expert services, reports, consultations, opinions, or expert testimony, depending on the experience of the engineer and the character, magnitude and importance of the work or subject involved, a charge of from $25 to $100 per day should be made; (2), for time of consulting engineer on designs and for more ordinary services, examinations, reports, etc., a charge of from $15 to $50 per day—seven hours of actual time to be considered one day, except that, while absent from home city or attending court, each day of twenty-four hours or part thereof shall be considered one day, irrespective of actual hours of time devoted to the case.

Additional charges should be made for all actual expenses and assistants. The cost of assistants should be determined by adding to the payment for wages 50 per cent for overhead charges.

GENERAL PROVISIONS

Traveling expenses and expenses involved in making borings, soundings or tests, or in the collection of similar data necessary for the proper designing or planning of the structure or project, should be paid for by the client in addition to the commission herein provided for, it being understood, however, that all ordinary measurements and surveys shall be considered a part of the regular work of the engineer, for which no extra compensation shall be expected.

When alterations or additions are made to contracts, drawings, or specifications, or when services are rendered in connection with legal proceedings, franchises or right-of-way, or failure of contractors, a charge based upon the time and trouble involved shall be made in addition to the commissions herein provided for.

It is recognized that in any work undertaken by an engineer there may be problems requiring the services of a specialist. The cost of such special assistance shall be paid by the client.

Drawings and specifications, by any method of payment for services, are to be considered the property of the engineer, but the client is entitled to receive one record copy of the same upon payment of the actual cost of making the copies if no duplicate set is at hand.

Good designing may be vitiated by poor construction, especially in any work which requires careful supervision and inspection during its progress, such for example as reinforced-concrete work. For this reason it is for the best interests of the client that the engineer be given full direction of the work during the construction period, or, if this is impracticable, that the client employ inspectors approved by the engineer.

Cost of Brick Roads in King County, Washington

VITRIFIED brick have been specified as road-surfacing material in King County, Washington, to the extent that when contracts now under way are completed the county will have about 18 miles of highway, varying in width from 16 to 20 ft., paved with vitrified brick. This is exclusive of the city of Seattle, which had, at the end of 1912, 702,169 sq. yd. of brick paving.

The necessity of extending the highway system which gives access to Seattle from the surrounding districts led to the voting of $3,000,000 worth of bonds, which were approved by the legislature in 1912, for the purpose of building permanent highways or trunk roads in King County. It is claimed that the success with vitrified brick of local manufacture as a road surfacing is such that a good deal more of the brick paving will be laid next season.

King County officials have expressed the opinion that the behavior of a brick pavement when first put in use will prove its quality, and if care is exercised to correct all defects which appear during the first few years of its life the maintenance cost thereafter, on a well constructed brick pavement, should be almost negligible. In accordance with this the various brick roads which have thus far been built in King County include in the contract a five-year guarantee, which is designed chiefly to protect the county pavements from inferior workmanship in foundation, base or grouting. The specifications accepted as standard in the county are much the same as those adopted by the American Society of Municipal Improvements and are as shown in the table.

COST OF BRICK ROADS IN KING COUNTY, WASHINGTON

Road	Location	Ready for traffic	Base	Width	Brick	Length in miles	Contract price, including grading	Cost per mile
Kent	South of Kent	Spring, 1913	5-in. concrete	16 ft. 10 in.	No. 2 brick	2.90	$58,750	$20,258
Puyallup	Puyallup	Spring, 1913	4-in. concrete	16 ft.	No. 2 brick	0.75	18,000	24,000
Kittitas Co	Ellensburg	Summer, 1913	4-in. concrete	16 ft. 10 in.	No. 2 brick	1.18	23,500	19,906
Snohomish Co	Stanwood	Autumn, 1913	6-in. concrete	18 ft. 10 in.	No. 2 brick	1.00	25,330	25,330
South Trunk	Auburn	Autumn, 1913	5-in. concrete	20 ft. 10 in.	Highway block	3.00	87,500	29,167
Bothell Road	Bothell	Early in 1914	5-in. concrete	18 ft. 10 in.	Highway brick	3.33	84,000	25,228
North Trunk	N. 85th St.	Early in 1914	5-in. concrete	18 ft. 10 in.	Highway brick	6.13	149,750	24,429
Meadows	South City Limits	Oct., 1913	Macadam	20 ft. 10 in.	Highway block	0.20	....	17,500*

*Estimated.

Proposed Tunnel Canals in Norway

A PLAN for a tunnel canal between Christiania and Lake Oieren in Norway is described in the engineering supplement of the London "Times." The lake is 325 ft. above sea level and is separated from the sea by a mountain 525 ft. high. It is proposed to construct a level tunnel 9 1/3 miles long from the north end of Lake Oieren to Lake Ostensjo, from which another tunnel 2 miles long would run to the Bakkelags height, whence four deep locks would lead down to the Christiania fjord. The long tunnel canal would be 30 ft. wide and 16½ ft. deep.

The variations of water level in Lake Oieren, amounting to as much as 10 ft., would necessitate a regulating lock at the entrance to the main tunnel. Lake Ostensjo is 13 ft. higher than Lake Oieren and would have to be lowered accordingly. The locks at the Bakkelags heights would be 200 ft. long and 26 ft. wide, with a fall for each of 82 ft. The cost is about $5,000,000.

Reclamation Service Expenditures for 1914

DURING the year 1914 $23,460,555 will be expended in the construction of the projects under the supervision of the U. S. Reclamation Service. This will exhaust the bond loan resources of the Reclamation fund, made available under the act of June 25, 1910. The money will be apportioned to the projects as follows:

Salt River project, near Phœnix, Ariz., $909,846; near Yuma, Ariz., $831,126, and near Orland, Cal., $255,700; Grand Valley project, near Grand Junction, Col., $1,873,184; Uncompahgre project near Montrose, Col., $750,861; Arrowrock Dam and other features of the Boisé project, Idaho, $3,600,057; Minidoka project, near Rupert and Burley, Idaho, $476,874; Huntley project, near Billings, Mont., $281,000; Milk River project in the region tributary to Great Falls, Mont., $1,393,083, with an additional sum of $843,630 for the St. Mary Storage feature; Sun River project, near Simms and Fort Shaw, Mont., $1,712,337; Lower Yellowstone project near Savage, Mont., $103,800; North Platte project near Mitchell, Neb., $733,875; Truckee-Carson project near Fallon, Nev., $807,706; Carlsbad and Hondo projects near Carlsbad, N. M., $304,000; Rio Grande project and the Elephant Butte storage works, $3,683,000; North Dakota Pumping project near Williston, N. D., $70,000; Umatilla project near Hermiston, Ore., $572,000; Klamath project near Klamath Falls, Ore., $175,219; Belle Fourche project near Black Hills, S. D., $163,039; Strawberry Valley project near Utah, $686,641; Okanogan project in the State of Washington, $149,000; several projects in the vicinity of North Yakima, Wash., $1,380,739; and Shoshone project near Powell and Cody, Wyo., $593,399.

Preliminary investigations and the inauguration of new reclamation enterprises in the West will involve the expenditure in Central Oregon and on the Columbia River of $472,916; $5,000 in California, $10,000 in the State of Washington on the Palouse project, $100,000 in Oklahoma in the vicinity of Lawton, $10,000 in Texas on the Pecos River, $50,000 for the exploration of the Colorado River basin and the determination of its resources in the matter of water storage and diversion sites, and $50,000 in artesian experiments, the location of which have not yet been determined. The remainder of the $23,460,555 will be appropriated for town-site development and other related works.

THE PENNSYLVANIA RAILROAD HAD 88,586 SHAREHOLDERS on Jan. 1—the largest number in its history—an increase of 13,431 for the calendar year 1913. Of this total 42,514, or 48 per cent, were women.

Sewage Treatment at Sturgis, Michigan

Tanks, Filters and Pumping Station Completed in Connection with New Separate Sewers in Manufacturing Town of 5000 People

A SEPARATE system of sewers, a sewage-pumping plant, and treatment works with a maximum daily capacity of 78,200 gal. have been completed at Sturgis, Mich., a rapidly growing manufacturing city of 5000, which five years ago owned no improvement except a water supply. The designer of the new works is George S. Pierson, consulting engineer of Kalamazoo. The following description of the new system was given in a paper presented before the recent annual meeting of the Michigan Engineering Society by H. C. Sherman, assistant to Mr. Pierson.

Contracts were awarded in June, 1912, and the system was completed in September, 1913. In designing the sewers the separate system was adopted throughout. About 2 miles of sewer, ranging from 18 to 27 in. in diameter, were built to take care of the storm water from the business district, as well as from streets that are to be paved in the near future.

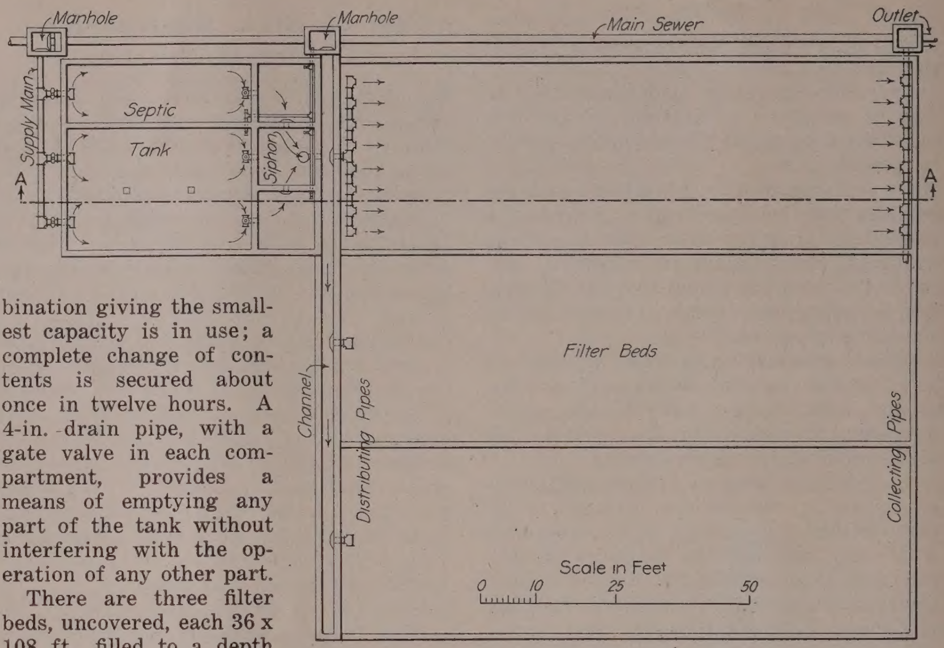
The sewers, including the outlet line, are built in sizes ranging from 18 in. in diameter in the main down to 8 in. in some of the laterals. Good grades were obtained in all parts of the city, and in no place did the excavation exceed 15 ft. No ground water was encountered in any part of the city, so no seepage was considered in calculating the capacity of the treatment works.

PUMPING STATION

In the western part of the city is a section so much lower than the rest of the city that it was necessary to install a pumping station to lift the sewage to a point high enough to be taken care of by the gravity mains. A pit 12 ft. in diameter and 16 ft. deep was built, into which is discharged the sewage from this district. In this pit were installed two 3-in. vertical centrifugal pumps, direct-connected with 5-hp. General Electric motors, having a speed of 1120 r.p.m. Each pump has a capacity of 300 gal. per minute against a 25-ft. head. When the sewage reaches the desired height the pumps are started automatically by a switch operated by a float, and are stopped in the same manner when the pit is emptied. The amount of sewage so pumped is only a small percentage of the total.

TREATMENT PLANT

The sewage treatment plant is of reinforced concrete. The settling tank is 36 x 48 ft., and is separated into five compartments, the two larger being 24 x 36 ft. and 12 x 36 ft. respectively, and each of the others 12 ft. square. The common depth is 8½ ft. Flow into the various compartments is governed by shear or gate valves. By various combinations of the compartments the capacity of the tank may be regulated to any of the following amounts: 31,600, 37,700, 46,600, 52,700, 58,800, 72,100, or 78,200 gal. At present the combination giving the smallest capacity is in use; a complete change of contents is secured about once in twelve hours. A 4-in. drain pipe, with a gate valve in each compartment, provides a means of emptying any part of the tank without interfering with the operation of any other part.



Plan of Sewage Purification Works at Sturgis

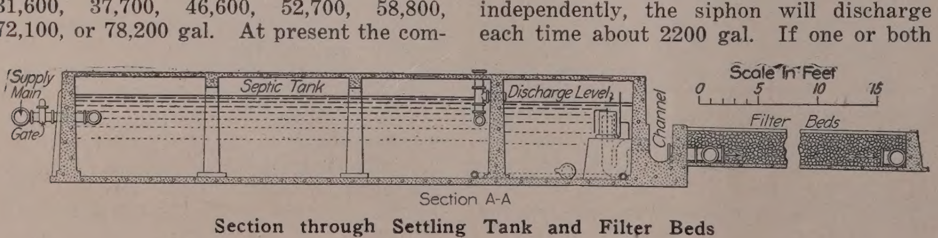
There are three filter beds, uncovered, each 36 x 108 ft., filled to a depth of 2 ft. 9 in. with crushed limestone from ¼ to 1½ in. in size. The beds have a slope of 6 in. from the upper to the lower end. Two filters will be in service at one time. At the end of each week the idle unit will be operated and one of the others laid off. The distributing and collecting pipes are made up of vitrified pipe tees, laid with open joints and ends.

In the first, or intake, manhole is a baffle of such height that in case of stoppage in the tank or siphon the sewage will overflow and pass out through the main sewer without flooding any part of the plant. In the second, or intermediate, manhole, is a shear valve that may be opened in case it is desired to use the septic tank but not the filters.

Sewage enters the first manhole and runs through the supply mains to the settling tank. The inlet and outlet pipes of the tank are submerged at all times to prevent, as much as possible, any agitation in the sewage and also to discharge from a level containing the least amount of suspended matter. The depth of sewage in the tank is about 7 ft., and at no time does the level of the surface fluctuate more than 1 or 2 in.

SIPHON FEED

From the first compartment the sewage is discharged into collecting chambers, or directly into the siphon chamber, through the pipes passing through the division wall. Most of the suspended matter is thus confined in the first compartment and the sewage in the collecting chamber shows comparatively litle color. When these chambers have filled to a depth of 6 ft. 9 in. the siphon automatically operates and the sewage discharges into the channel between the tanks and the filter beds until the level in the siphon chamber is lowered 29 in. Then the siphon remains out of action until sufficient head is reached to bring it into operation again. If the siphon chamber is used independently, the siphon will discharge each time about 2200 gal. If one or both of the collecting chambers at each side of the siphon chamber are used, however, the amount discharged will be approximately two or three times as great and the time between discharges correspondingly longer.



Section through Settling Tank and Filter Beds

As the channel fills the sewage passes through the openings to the filter beds, thence through the distributing pipes, and rises through spaces between the particles of stone. After the discharge of the siphon ceases the sewage gradually recedes, leaving the organic matter on the surface of the particles exposed to the oxygen present. The intermittent discharge of the siphon thus gives the beds alternate periods of flooding and rapid aëration. After passing through the filter beds the effluent is carried to the manhole and thence through the main sewer to the county drain.

COST OF ENTIRE SYSTEM

The cost of construction of the sewers and treatment plant was approximately as follows: Sewers, $50,200; pumping station, $1720; treatment plant, $9100, making a total of $61,020.

All work was paid for by general tax, except the sewers, which were financed partly by a special assessment of 33 1/3 cents per foot of frontage and partly by general tax.

Although the filters have not been in use long enough to demonstrate their efficiency, the report on samples of the effluent obtained in December and analyzed by the State Board of Health was favorable.

Sewers and pumping station were built by F. M. Benner & Company, of Marion, Ind., and the purification works by Andrew Geeck, of Owosso, Mich.

COAL CONVEYORS IN CHINA are described at some length in a recent issue of the "Engineer." In the province of Pe-chi-li, in the area commanded by Pekin and Tientsin, a Chinese federation, the Salt Guild has obtained a monopoly, and has adopted an aerial ropeway system developed by Bleichert's Aerial Transporters, Ltd., of London. The wire ropeway has a total length of about 15 miles, including a stem of 5½ miles, and two branches of 5 and 4½ miles respectively. The lines traverse numerous peaks, the total rise from Toli, the railway terminus, to the end of one of the branches being 1620 ft.

Chlorination Recommended for Buffalo

Temporary Treatment of Lake Erie Water to Be Succeeded by Filtration

THAT filtration should eventually be employed for the water supply of Buffalo, N. Y., and that sterilization by liquefied chlorine gas or hypochlorite of lime—to eliminate the occasional danger of disease-producing bacteria—be employed at present are the recommendations of Rudolph Hering, consulting engineer, of New York, in a report submitted Jan. 24 to Col. Francis G. Ward, commissioner of public works. Dr. Hering urged that the city at once proceed to buy land for the filtration plant that will inevitably be required, and that any temporary measure be so chosen and administered that it may be economically incorporated in the permanent works.

INTERMITTENT TURBIDITY

The new intake, to which the report has reference, is at the head of the Niagara River and the foot of Lake Erie, in about 24 ft. of water. While the water is entirely unobjectionable for most of the year, it is not perfectly clear for about 90 days, and is even turbid for three or four weeks, due to waves which sometimes disturb the lake water to a depth of 30 ft., taking up fine particles of clay and sand, chiefly the former.

This clay and sand are not unhealthful, states the report, the amount of mineral matter represented being so small that its effect upon the human body may be ignored. The mineral matter in the unfiltered supplies from the muddy Missouri and lower Mississippi rivers, greatly in excess of that from Lake Erie, has not been known to cause disease, however unpleasant its consumption may be.

Objection had been made, however, to this occasional turbidity and fear had been expressed that with the stirring up of the bottom by the waves some pathogenic bacteria deposited near the shores might also be included in the water furnished the city, although Dr. Hering found no evidence as to the existence of pathogenic bacteria in the water that was being supplied to the city, either at times when it was clear or when it was turbid.

The typhoid-fever rate has been practically reduced one-half since the introduction of the Emerald Channel supply, the present rate comparing favorably with the lowest in American cities. It is, nevertheless, desired to obviate the remotest danger to the citizens by adopting a treatment of the water at times when any doubt may exist.

Of the two methods of removing turbidity, filtration and sedimentation, the latter, though far less expensive, would give an unsatisfactory efficiency, according to Dr. Hering, and would not be worth the expenditure. The finest particles of clay would not be removed even after a full day's settlement. Settling basins would have to be very large on account of the slowness of the movement of depositing fine clay. The minute particles of sand, however, deposit more rapidly and could more readily be removed by sedimentation. A large percentage of pathogenic bacteria would also be removed by being carried down with the settling particles of clay and sand, to which they gradually attach by attraction.

Filtration causes all the suspended matter, and with it the microscopic bacteria which attach to it, to be removed. The result is both a clear and healthful water. But it has been estimated that the cost would add about 50 per cent to the present water rates. The other method of bacterial removal—sterilization—which is much less expensive, does not in the least reduce the turbidity of the lake water, although it would render its consumption healthful by removing any danger of infection.

Unless the additional cost of filtration is willingly paid by the people, for the sole purpose of having clear water at all times, sufficient protection against disease may, in Dr. Hering's opinion, be obtained by sterilization. Chlorination, by the use of either hypochlorite of lime or liquid chlorine gas, involves a small expense for installation. The chlorine process would probably be continued when filtration is introduced, because it is much more effective, at a given cost, in the destruction of pathogenic bacteria.

The report points out that determinations of the composition and quantity of the suspended matter in the lake water of the Emerald Channel at various times, as well as the amount of chlorine required by the excess clay, should be made as soon as possible. The city is advised to ask for detailed bids covering building and operating costs by both methods of sterilization, insisting on a guarantee of bacterial removal per unit of chemical.

Dr. Hering concludes:

"It is my opinion that sterilization by either method will safeguard the public health at a very much lower cost than the adoption of filtration at this time. As soon as the city is prepared to pay the cost of giving the water at all times an excellent appearance, filtration is the system to be adopted."

Reports on New York Barge Canal

Consulting Engineers Commend Execution of Work and Explain Why Total Cost Is Likely to Exceed Original Estimate

THAT the New York State Barge Canal work has been well planned and executed, that the cost of the construction work will come within the original estimate for the construction, that the elements which will raise the total cost above that of the estimate could not have been foreseen and that the legal obstacles standing in the way of completion of the railroads should be overcome as speedily as possible are the concurring views of Col. Thomas W. Symons and Col. E. W. Van C. Lucas, consulting engineers to the Canal Board, as expressed in their reports recently submitted to the board.

Throughout the year, thinks Colonel Symons, the work has progressed well along all lines. About $64,000,000 worth has been completed, about $20,000,000 worth is now under contract and about $6,000,000 worth is yet to be contracted for. This would make the whole Erie, Oswego and Champlain Barge Canal work, so far as construction is concerned, cost about $90,000,000, which is deemed remarkably close to the estimate.

ITEMS NOT ORIGINALLY ESTIMATED

While this is true about the construction, he points out that there are various elements which will possibly raise the total cost of the project to a sum considerably greater than the original estimate. These elements are enlargements and changes ordered by legislation, high prices which had to be paid for the necessary land, excessively high judgments secured and claimed by people for damage to water-power rights and inconveniences to their business, damages claimed by contractors on various grounds and delay and high charges of railroads in providing crossings for the canal. In all of these matters Colonel Symons thinks it was impossible to make accurate preliminary estimates.

RAILROAD CROSSINGS

With good fortune, and the railroad-crossing problem quickly solved, he thinks the construction work should be done in about two years more. He urges, however, that arrangements be made with the least possible delay for the construction of these crossings. Failure to supply these crossings has prevented and will prevent the completion of several contracts, has cost and will cost the State a large amount of money—probably as much as the cost of the crossings themselves—and will delay the completion of the canal and its use for commerce, keeping the canal interests of the State from reaping the advantages for which the canal was designed.

The attention of the Canal Board is invited by Colonel Symons to the desirability of making or encouraging an investigation into the character, size, construction, method of propulsion and cost of boats and barges for use on the canal, so that those preparing to enter into the business of transportation will be able to get boats ready for use when the canal is completed.

Colonel Lucas believes that the construction work can be completed during the season of 1915, provided that the legal questions affecting the railroad crossings are speedily settled; that otherwise there will be an indefinite delay, at a cost to the State, without any adequate return, of more than $4,000,000 annually.

HIGH-GRADE ENGINEERING FORCE

He considers that from an engineering standpoint the work is being cared for properly and under competent supervision, and that the engineering force is of a high grade. Quoting him, "The canal is being built within the estimated cost of construction and engineering, which I consider a fine achievement at any time for work of such magnitude and complexity, and a remarkable one in view of the increased cost of labor and materials since the estimates were made, and the extensive additions to the plans as originally proposed, required by acts of the legislature and resolutions of the Canal Board, without corresponding appropriations. . . . I desire to impress on the board the fact that the excess in the expenditures which is likely to occur is in items which have not been seriously considered heretofore, or which have been omitted entirely from classifications made and submitted to the public, giving what purported to be the entire cost of the Barge Canal project."

RAPID TIMBER ERECTION has been the construction feature of the framework for the Palace of Food Products at the Panama-Pacific Exposition, San Francisco. Beginning Oct. 1, 1913, 900,000 ft. of timber in framework were erected in thirty days, thus completing 45 per cent of the contract for the building in one month.

Sewage Treatment by Aëration and Contact in Tanks Containing Layers of Slate

Lawrence Experiments, If They Can Be Applied on Large Scale, Suggest Important Advances in Sewage-Treatment Practice

By H. W. CLARK and GEORGE O. ADAMS
Chemist / Assistant Chemist
Massachusetts State Board of Health

PARTIAL purification, or even so-called complete purification, of sewage may be accomplished upon much smaller areas than have hitherto been believed possible and at much lower cost if the results of two years' experimental work at the Lawrence Experiment Station are found to be applicable on a large scale.

In the Engineering Record of June 28, 1913, page 715, an article was presented by us describing some experiments upon the aëration of sewage, especially in tanks containing layers of slate, and the results of this treatment as an aid to increased rates of filtration were described. It was stated briefly in that article that the great changes in sewage resulting from treatment in the Lawrence tank were due to aëration combined with contact of the sewage with growths upon the slate. These growths and the resulting clarification of the sewage occurred under aërated conditions—with dissolved oxygen present in the sewage and circulation of the sewage through the tank caused by the currents of air.

While aëration of sewage has been investigated from time to time at the Lawrence Experiment Station, this most recent work was begun nearly two years ago, and as it has progressed it has become evident that under the conditions prevailing in this Lawrence tank a considerable purification of sewage can be obtained without filtration when the tank is constructed and operated as described in our previous article. This purification takes place, moreover, both when stale sewage, such as comes to the experiment station, is treated and in fresh sewage before much decomposition has set in. This has been proved by our experimental work.

PERIOD OF AËRATION

The requisites for such purification are dissolved oxygen, circulation of the sewage for a certain period of time, and sufficient surface of slate or other material upon which abundant growths may occur. The slate walls in the Lawrence tank are at the present time vertical and about 1 in. apart, and only about three-fourths of the tank contains them. The tank has been operated daily for twelve months, and the time of aëration of the sewage has been reduced to five hours for each volume treated. This means, of course, that with a tank holding 1,000,000 gal., from 4,000,000 to 5,000,000 gal. of sewage could be treated each day. Clear sewage can be drawn from the tank as soon as the aëration is stopped, as by that time nearly all the suspended matter has been collected by the growth on the slates.

The last inch or so in depth of sewage contains all the sludge which may have become detached from the slate and its suspended matters are easily separated from the liquid. This sludge is practically odorless, has lost all offensive characteristics common to sewage sludge, and earlier in the experiment, when the period of aëration was ten hours or more, the aërated sewage was frequently non-putrescible, as shown by incubation, and even with the comparatively short period of aëration now given the sewage odor has disappeared.

A considerable portion of the increase in stability of sewage gained during this treatment is, however, due to coagulation and clarification rather than to absorption of dissolved oxygen, as shown by the fact that samples of the raw sewage, such as were treated in this tank, when diluted with distilled water and incubated five hours (the time of aëration in the tank) at the same temperature as the sewage in the tank (60-65 deg. Fahr.), absorb less than 1 part of oxygen per 100,000 parts. This is not a sufficient amount to account for the purification occurring in the tank.

AMOUNT AND COST OF AIR

Air is used at present at the rate of 25,000 cu. ft. per hour per 1,000,000 gal. of sewage 5 ft. in depth. With better methods of distributing the air this volume of air might be reduced or the volume of sewage considerably increased. At present the air is applied through a perforated pipe at the bottom of the tank. The cost of power alone for such aëration, on a basis of 4 cents per kilowatt-hour, as estimated by ourselves and by an expert of a prominent engineering firm, would be less than $2 per 1,000,000 gal. for 5 hours' aëration, using 25,000 cu. ft. per hour. These figures of costs are about three-fourths as high as those of Black and Phelps given in their report upon "Treatment of Sewage Entering New York Harbor." The work of Black and Phelps and others, moreover, was simply upon aëration, while the treatment in the Lawrence tank not only aërates the sewage, but, as stated previously, collects much suspended matter, and, what is more important, coagulates much of the colloidal matter present in the sewage. The matters removed from the sewage by the action in the Lawrence tank collect upon the slates in jelly-like masses, often ⅜ in. or more in thickness, slough off from time to time and are easily removed from the bottom of the tank.

PREVIOUS INVESTIGATIONS

In previous years much work has been done at Lawrence and elsewhere upon the aëration of sewage. The late Col. George E. Waring probably did the first work in America upon a working scale along this line, and constructed at Wayne, Pa., somewhat more than twenty years ago, a sewage-disposal system which included aëration as one of its important features. At these works, as then operated, the sewage, after passing through screens, entered a receiving reservoir with a capacity of 90,000 gal. Its treatment in this reservoir was as follows:

"Two Barr duplex pumps, with a capacity of about 22,000 gal. each per hour, or 525,000 gal. per day, force the sewage up the hill on the left of the creek to the distributing wall. This 12-in. force main is of spiral weld-steel pipe, is 480 ft. long, and has a rise of about 100 ft. The lower end of the force main was placed above the ground to obtain a grade that would allow it to drain dry through the aërating pipe mentioned below. Both pumps are started when the screening reservoir is nearly full, and, as designed, the sewage is first delivered back into the receiving tank through a 4-in. aërating pipe, the object being to deodorize the sewage and increase its oxygen. Aëration is maintained for from 60 to 90 minutes, after which the valve in the aërating pipe is closed and the sewage is delivered into the well." (Rafter and Baker, 1894, "Sewage Disposal in the United States," page 535.)

TREATING SEWAGE BY AËRATION

A series of experiments on the treatment of sewage by aëration was made for the Metropolitan Board of Works, London, by Dr. A. Dupre and W. J. Dibdin at about this period, and the results are given by Rafter and Baker. A table is given showing the analyses of the sewage before and after aëration, and the statement is made that it is evident from the analyses that "aëration had but little effect upon the sewage."

The same conclusion was reached in the experiments by S. K. Hine and are recorded in a paper entitled "Note on the Practical Observation of Organic Matter in Water," by Prof. W. P. Mason and S. K. Hine ("Journal" of American Chemical Society, Vol. 14, page 7). These experiments were presented as a graduating thesis at Rensselaer Polytechnic Institute in June, 1892. In these experiments "mixtures of sewage and water were placed in tin cans or glass-stoppered bottles, the receptacle being not over half filled, and the can or bottle was then fastened to the connecting rod of a horizontal steam engine of 10-in. stroke and the engine run at 75 r.p.m., so that in an hour the receptacle was shaken 9,000 times and traveled 1.25 miles. The samples were shaken from 13 to 60 hours and twenty-four samples were tried, analyses being made before and after shaking."

The paper states that "examination and results show that the amount of oxidation which took place during agitation of the water was very trifling, a finding entirely in accordance with Prof. Lee's observations of the waters of the Niagara River before and after passing Niagara Falls. Direct oxidation does not seem to be a factor of any considerable importance in the purification of polluted waters."

Dr. T. M. Drown, for many years chemist to the Massachusetts State Board of Health, after experiments with natural waters and with water to which a small amount of sewage was added, concluded that "the oxidation of organic matter in water is not hastened by vigorous agitation with air or by air under pressure."

The object of these early studies was, as the quotations show, the oxidation of organic matter by the air with which it was agitated or placed in contact, and as these matters were changed little, if any, by such aëration, as shown simply by ammonia determinations, the process was considered to be ineffective and a failure. Similar results were obtained in this laboratory on aërating sewage in clean bottles, and it was only when the bottles became coated inside with a "growth" that any appreciable clarification was obtained.

Notable studies upon aëration have in recent years been made by Black and Phelps and described by them in their report mentioned above. In all their work, however, as in the previous studies mentioned above, aëration alone was tried, and the object of their study was to observe and note the changes induced by passing certain volumes of air through certain volumes of sewage. By such aëration Black and Phelps showed that certain changes in sewage occur which increase its stability or rather free it more or less from its unstable, quickly putrefying elements.

RESULTS OF EXPERIMENTS

The present work at Lawrence, however, is essentially different from any of the preceding work along this line and results in a substantial purification. A summary of the aëration work at Lawrence during the last six months shows the following results:

1. The time of aëration has been five hours, and from July 1 to Sept. 1, 1913, air was applied at the rate of 100,000 cu. ft. and thereafter at the rate of 25,000 cu. ft. for sewage 5 ft. in depth. About July 1 the slates were changed from a horizontal to a vertical position. At that time there was from ½ to 1 in. in thickness of growth and sludge on the slates. This matter was fermenting but inoffensive.

2. The comparative clarification by settling tanks, by chemical precipitation and by the Lawrence aërating and coagulating tank is shown in Table 1. The settled sewage was given a period of twelve hours for sedimentation, the precipitated sewage received 1000 lb. of aluminium sulphate per 1,000,000 gal. and four hours for subsequent sedimentation. The treatment in the slate tank was for a five-hour period. The average of six months' results is given in the table, and the per cent reduction refers to the removal of matters present in the raw or untreated sewage. It will be noticed that 87 per cent of the total suspended matters have been removed by the aërating-tank treatment and that the removal of organic matters in solution and semi-solution (colloidal matter) is three times as great by this treatment as by prolonged sedimentation.

TABLE 1—COMPARATIVE RESULTS OF SEDIMENTATION, CHEMICAL PRECIPITATION AND LAWRENCE TANK TREATMENT

	Per cent reduction of			
	Albuminoid ammonia		Solids	
	In suspen.	In sol.	Total suspen.	Volatile in sol.
Settled sewage.........	52	16	46	8
Precipitated sewage.....	68	28	76	24
Aërated sewage — Lawrence tank	80	28	87	24

3. The rates of the trickling filter to which aërated sewage was applied were as follows, in gallons per acre daily: July 1, 8,000,000; Aug. 1, 8,500,000; Aug. 10, 9,000,000; Oct. 1 to date, 10,000,000. The filter is 10 ft. deep and contains crushed stone passing a 2-in. ring and held back by a 1-in. ring.

TABLE 2—AVERAGE ANALYSIS FOR SIX MONTHS OF RAW SEWAGE, AËRATED SEWAGE AND EFFLUENT FROM TRICKLING FILTER (PARTS PER 100,000)

	Ammonia			Kjeldahl Nitro.		Nitrogen as		Oxygen Consumed	Chlorine
		Albuminoid							
	Free	Total	In solution	Total	In Sol.	Nitrates	Nitrites		
Raw Sewage................	3.62	.57	.32	1.04	.62	....	....	3.52	13.60
Aërated sewage..............	3.28	.28	.23	0.49	.43	....	....	1.89	12.50
Effluent from trickling filter..	1.25	.12	.08	0.28	...	1.78	.0523	1.35	12.90

The effluent from the trickling filter has been stable at all times, and clear after the typical brown sediment has settled.

CONCLUSIONS

It is evident from the results of this work now covering two years' observation and experiment that there is much of promise in it; that if successful upon a large scale a partial purification, or even so-called complete purification, of sewage may be accomplished upon much smaller areas than has hitherto been believed possible, and also at a much lower cost. The cost of power for aërating and coagulating the sewage in the Lawrence tank should not average more than from one-third to one-fifth as great as the cost for chemicals in chemical-precipitation treatment, and the aërating tank produces a somewhat better purified effluent. The sludge liquor in the tank is also much less in volume and more easily handled than the sludge liquor from chemical precipitation. It is possible, moreover, that agitation and aëration in this slate tank might be accomplished more cheaply than by the method used in these experiments. It is also evident, as described in our previous paper, that the effluent from this tank can be purified upon trickling filters at a much higher rate than can sewage from settling or chemical-precipitation treatment. This is probably due partly to the better clarification of the sewage and partly to its decreased oxygen requirement. The fact, also, that the sewage after treatment has lost all offensive odor is of great importance, assuring purification without nuisance.

Purifying Water in an Indoor Swimming Pool

Sanitary Study of Results with Hypochlorite in 55,000-Gallon Tank

THE great expense incurred in changing the water in indoor swimming pools is the most important consideration that has led to the installation of purification systems whereby water is returned to the tank after filtration. John F. Norton, of the Massachusetts Institute of Technology, in an address before the American Public Health Association at the Colorado Springs convention, described a sanitary study which he had made of the purification system at the Harvard tank of the Cambridge Young Men's Christian Association. An abstract of the paper follows:

The pool is 60 x 20 ft., the greatest depth being 7 ft., giving a capacity of 55,000 gal. Water is supplied from a 400-ft. artesian well at the rate of 50 gal. per minute. In the cleansing process the water is pumped through a closed filter 6 ft. high and 4½ ft. in diameter filled with quartz pebbles.

HYPOCHLORITE TREATMENT

In front of the filter is a bypass, controlled by a needle valve leading through a cylinder containing sulphate of alum, which mixes in the proportions from 1/5 to ½ grain per gallon. At the cessation of filtration each night 1 lb. of hypochlorite of lime is dissolved in the water. In the morning the sides and bottom of the tank are cleaned by a suction apparatus.

Bacteriological and chemical tests were made, using samples from the middle of the tank. No hypochlorite was used for the first two days, during which 125 swimmers used the pool, but it was used during the remaining seven weeks through which the tests ran. Samples were taken just before the pool was thrown open to bathers.

The results of the physical examination can be briefly dismissed. At no time was any odor or color in the water observed, except for a slight odor of chlorine once noticed when a sample was collected. No complaint from this point was ever received from any of the swimmers. The water was clear and free from sediment except for a short period when particles of alum were noticed in the tank.

SWIMMING POOL INCUBATOR FOR BACTERIA

A study of the bacterial and chemical data shows that a swimming pool is an ideal incubator for bacteria, particularly those coming from the body, on account of the favorable temperature and the presence of food material. Addition of hypochlorite causes an immediate reduction in the number of bacteria, which is kept within reasonable limits by this and the rest of the purifying system for about four weeks. The same is true of gas-forming organisms. To Mr. Norton's mind the two main factors in keeping the bacterial count low are the addition of the hypochlorite and the sedimentation which takes place during the night when the tank is out of use, this latter being followed by efficient cleaning. After about four weeks practically all of the colonies on the plates were of a yellow chromogenic bacillus, which was not further identified. Mr. Norton believes that this organism gradually developed a resistance to hypochlorite and could be eliminated only by emptying and cleaning the tank. This step made it impossible to follow up the reduction in the number of bacteria noticed in the last sample taken. An attempt was made to isolate the staphylococcus from some colonies on the agar plates, but in only one instance was this successful.

Chlorides are not an index of the pollution of a swimming pool, as the variation from the original water was only slight and did not correspond with the bacterial results; this has been noticed by others. The determination of the free ammonia gives a partial but not entirely satisfactory index of the gas-forming organisms present.

The best index of the bacterial pollution, concludes Mr. Norton, is given by the nitrite determination. This relation between nitrites and total count of bacteria is shown very clearly if the results are plotted. The mathematical relation—nitrogen as nitrites $\times$ 1,000,000 = total bacteria—may have been only a coincidence.

THE UNITED STATES GEOLOGICAL SURVEY printed 253,850 copies of reports during the fiscal year ended June 30, 1913; the number distributed, including previous reports, was 375,213. The effect of the reduced rates on maps, resulting from the damage caused by the fire in the Survey office in May, is shown by the 27,000 geologic folios sold between May 20 and June 30.

The Functions of the Engineering Society

Resume of Address Delivered at Illinois Society of Engineers and Surveyors by L. K. Sherman, Engineer and Contractor, of Chicago

THE time is ripe when engineering societies should enlarge their field of endeavor from the purely technical and literary functions to that of engaging directly in promoting the material status of the profession and individual members. This result can be accomplished only through organization; individual effort fails. In business, consolidation alone survives. If engineers are to progress and to retain their rightful sphere they must do so through organization. The legal and medical professions are strongly organized to guard and advance their interest in legislation and in public relations. The architects of Illinois have an able organization, which legislated a certain line of structural work out of the engineers' hands into theirs.

However much the engineer may disagree with the methods of the building trades union, he cannot but admire the efficiency of the organization. Mr. Sherman was careful to state that he did not mean to undertake the desirability of maintaining the dignity of engineering as a profession, but dignity, he believes, should not interfere with business progress in engineering, any more than it does with the law or medicine.

As an indication that the present engineering societies are not entirely meeting the problems of material professional welfare, mention was made of the formation of the American Institute of Consulting Engineers in New York to handle these matters, and the recent and most radical development, the formation of the Chicago Technical League, a union of engineers affiliated with the American Federation of Labor.

Among the plans, projects and suggestions for advancing the status of engineers are: Licensing engineers; civil service laws; legal compensation for city engineers; trade unions; codes of professional ethics; standardization of fees and employment bureaus operated by the societies. Mr. Sherman did not attempt to argue for or against these plans, but advocated that it is the duty and function of the engineering society to consider such plans and to have an organization with the necessary executive machinery to act for those measures which are of benefit and to circumvent those detrimental to the engineer and his allied works. The local or State society is the logical organization to initiate these material betterment measures and the local efforts should be supported in turn by the national societies.

Along the above line Mr. Sherman reported that Walter A. Shaw, engineer member of the new Illinois Public Utility Commission, assured him that the action of the society in presenting a petition to Governor Dunne urging the appointment of an engineer on the commission had the desired effect. Mr. Sherman is of the opinion a member seeking an appointment might be endorsed by a committee as one whose qualifications fitted him to hold the position sought in case the committee found the applicant worthy. In case it did not so find, then nothing would be done and no one outside of the committee need know of it.

Annual Costs of Treated and Untreated Piles in Trestles

Figures Deduced to Show Probable Saving per Year to Southern Pacific Company in Treating 105,000 Douglas-Fir Piles

THE annual cost of structures having a life of from five to thirty years and a first cost of from 10 to 60 units (dollars or cents) is given in tabulated form in the report of the committee on preservation of timber presented at the recent convention of the American Railway Bridge and Building Association. Utilizing this table, which is applicable to structures of various types, the committee analyzes the saving in the use of treated piles for trestles on the Southern Pacific.

The Southern Pacific, according to the report, has about 105,000 creosoted Douglas-fir piles in trestles. They were treated by the boiling process, and range in age from one to twenty-three years, with probably more than two-thirds of them over twelve years old. Of this number not more than 500 have ever been replaced on account of decay, and many of those twenty years old are as good as when driven. The committee does not doubt that they will be good for at least thirty years, and estimates the life of the same timber untreated as eight years.

ANNUAL COST OF STRUCTURES, BASED ON LENGTH OF LIFE

First	Length of life in years																	
Cost	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	25	30
10	2.31	1.97	1.73	1.55	1.41	1.29	1.20	1.13	1.06	0.99	0.96	0.92	0.89	0.85	0.83	0.80	0.71	0.65
20	4.62	3.94	3.46	3.09	2.81	2.59	2.41	2.26	2.13	1.97	1.92	0.84	1.77	1.71	1.65	1.60	1.42	1.30
30	6.93	5.91	5.18	4.64	4.22	3.88	3.61	3.38	3.19	2.96	2.88	2.77	2.66	2.57	2.48	2.41	2.13	1.95
40	9.24	7.88	6.91	6.19	5.63	5.18	4.81	4.51	4.26	3.95	3.84	3.69	3.55	3.42	3.31	3.21	2.84	2.60
50	11.55	9.85	8.64	7.74	7.04	6.47	6.02	5.64	5.32	4.94	4.81	4.61	4.44	4.28	4.14	4.01	8.55	3.25
60	13.85	11.82	10.37	9.28	8.44	7.76	7.22	6.77	6.38	5.92	5.77	5.53	5.32	5.13	4.96	4.81	4.25	3.90

Assuming these piles to average 40 ft. in length, each pile, at 10 cents per foot, would cost $4. The driving would cost $6, making the total of $10 per pile. If such a pile would last eight years, its cost, as shown in the table, would be $1.55 per year. Similarly, a 40-ft. creosoted pile, at 30 cents per foot, costs $12, which, added to the $6 for driving, makes a total of $18. If the pile will last thirty years, the annual cost will be $1.17. The difference is 38 cents per pile, or nearly $40,000 per year for the entire 105,000 piles.

Filter Operating Costs at Toronto

Average for Year Shows Expense of $1.037 per Million Gallons

THE slow-sand filtration plant at Toronto is situated on a low gravel island in Lake Ontario, 2 miles south of the city waterfront. There are twelve filter beds, each 117 x 312 ft., and a pure-water reservoir, 312 ft. square. The plant was designed to filter 57,600,000 U. S. gal. of water daily, necessitating a rate of flow of 6,000,000 gal. per acre.

Costs of maintenance and operation have been kept from the first and are presented in the following table, which is a rearrangement of data presented by Mr. Francis F. Longley before the Canadian Society of Civil Engineers. The method of operation during the winter involves the storage of several thousand cubic yards of sand, to be washed the following summer. For that reason the figures for September, October and November are nearer the normal average than that given for the twelve months; the average for those three months has therefore been added.

COST OF FILTER OPERATION AT TORONTO PER MILLION U. S. GALLONS

	Office and laboratory	Low-lift pumping to the filters	Filter-cleaning and sand-handling operations	Care of grounds and miscellaneous	Total cost per million gallons	
					Filtering	Filtering and pumping
1912						
January	$0.311	$1,720	$0.319	$0.142	$0.772	$2.492
February	0.302	1.493	0.367	0.132	0.801	2.294
March	0.381	1.563	0.521	0.147	1.049	2.612
April	0.407	1.394	0.382	0.171	0.960	2.354
May	0.362	1.239	0.592	0.190	1.144	2.383
June	0.332	1.422	0.559	0.096	0.987	2.409
July	Filters out of service					
August	Filters out of service					
September	0.412	1.809	0.822	0.045	1.279	3.088
October	0.363	1.667	0.756	0.048	1.167	2.834
November	0.453	1.546	0.638	0.138	1.229	2.775
December	0.415	1.246	0.473	0.181	1.069	2.315
1913						
January	0.289	1.382	0.697	0.064	1.050	2.432
February	0.250	1.282	0.645	0.051	0.946	2.228
Average for 12 months	0.356	1.480	0.564	0.117	1.037	2.517
Average for September, October and November	0.409	1.674	0.739	0.077	1.225	2.899

Proposed Long-Span Canadian Drawbridge

THE proposed bridge across the Second Narrows of Burrard Inlet, Vancouver, B. C., has been designed by Sir John Wolfe Barry, Lyster & Hartley, of London, associated with Cleveland & Cameron, of Vancouver. The bridge is located in a rather difficult position, where the low and high-water widths of the channel are 1030 ft. and 3248 ft. respectively, with a tide of 13 ft., a current of 7 knots, and an ordinary maximum depth of 96½ ft. On one side the foundations will be on soft sandstone overlaid with gravel and boulders, and in some places the foundations will have to be carried down to a depth of 100 ft. in gravel. The alignment will be curved over the tidal flat and the grade will rise to a clearance of 45 ft. over the channel. The bridge will carry one steam-car track, two electric-car tracks, one 37½-ft. roadway and one 8-ft. cantilever sidewalk.

Over the channel there will be a turntable drawspan of 581½ ft. center to center of end bearings, and there will be two 232-ft. fixed spans—all with riveted trusses 59½ ft. apart on centers. The turntable will be supported on four braced steel cylinders 49 ft. apart longitudinally and transversely, and the other piers will have riveted columns supported on pairs of steel cylinders. Six additional steel cylinders will also be sunk in the channel parallel to the open position of the bridge to carry the fender construction.

Rainfall, Sewage, Storm and Tidal Water in New York Harbor

Abstract of Report by Metropolitan Sewerage Commission of New York on Relations between Volumes to Be Provided For in Sewage Treatment and Drainage

IN its ninth preliminary report the Metropolitan Sewerage Commission of New York discusses the bearing upon the question of sewage treatment of the volume, intensity and frequency of rainstorms.

With respect to volume, the water which falls in the built-up parts of the city during the course of a year in the form of rain and snow and flows away to the sewers bears a very small proportion to the quantity of domestic sewage. The first effect of a storm is to increase both the volume and concentration of the sewage, and this effect depends on the violence and duration of the storm.

ALLOWANCE FOR STORM WATER

In the studies made by the commission for main drainage and disposal works for New York, allowance has been made for storm water by providing for twice the average dry-weather flow. This provision gives room for the maximum discharge of domestic sewage, which may be 50 per cent above the average at certain hours of the day and makes it possible to deal with a considerable amount of storm water, provided it falls at a time when the ordinary domestic flow is not at a maximum.

The importance of giving careful consideration to the rainfall is greater in designing collecting systems of sewers than in providing for final disposition. The function of such sewers is not only to carry off the drainage of the houses, but to prevent accumulations of water in the streets. It sometimes happens, when excessive falls of rain occur, that sewers are surcharged, thus interfering with the drainage of houses. It is usually impracticable to provide combined sewers of a size and grade sufficient to carry away the water which falls in storms with sufficient promptness to insure that inconvenience from flooding shall never occur. At long intervals rainfalls of exceptional severity take place, and to provide for these, sewers would have to be built so very large that they would represent a considerable investment over the sum required to give them sufficient capacity for all the ordinary and most of the heavy rains which are likely to fall.

Studies have been made by the commission to show the amount of storm water entering New York harbor from various parts of New York City. The rainfall was taken from the U. S. Weather Bureau records from 1875 to 1912, inclusive. The yearly average was 43.97, the minimum 35.73 and the maximum 58.68 in. The runoff was assumed to be from 40 to 80 per cent, according to the density of population of the various parts of the drainage areas.

It is shown that the percentage of daily rainfall to daily sewage is about 7 per cent in the most densely populated parts of the city. In the course of a day or month or year there is about ten times as much domestic sewage as storm water entering directly to the Lower East River. It was observed that the annual rainfall has had a wide range in the 37 years for which records are available. The minimum of 35.73 in., which is about 81 per cent of the average, occurred in 1895 and the maximum of 58.68, or about 133 per cent of the average, occurred in 1889.

INTENSITY OF RAINFALL

Marked variations have been observed in intensity of rainfall. In September, 1882, 14.51 in. fell, or about one-third of the total annual average. On the twenty-third day of the same month 6.17 in. fell in 24 hours, or 42½ per cent of the monthly rainfall in one-thirtieth of the month.

The records of the Weather Bureau have been examined since observations were begun for the heaviest rainfalls which have occurred in brief intervals of time. The intensities lasting 5, 10, 15 and 30 min. and 1 and 2 hours have been taken from these records.

Rates of precipitation have varied from 1 to 9 in. per hour. The greatest rate was 8 in. per hour and this was continued for 5 min. To facilitate the use of the data from the records of the U. S. Weather Bureau curves have been prepared to show the most important facts with regard to the intensity of rainfall. Fig. 1 gives the frequency of rates of different intensities for various periods of time, arranged according to the percentage of months in which they have occurred.

Fig. 2 shows the frequency of various intensities which have lasted for 5 min., arranged so as to indicate the number of times which they have occurred per year and the mean interval in months which has occurred between storms.

A rainstorm of unusual severity occurred in New York on Oct. 1, 1913. The storm was characterized by low rates of precipitation from 7 a. m. until about noon, when the severity increased until between 2 and 3 in the afternoon a maximum was reached. After 4 o'clock the rate of rainfall was comparatively insignificant. During the preceding day the weather had been cloudy with slight precipitation. At its height there was a period of 5 min., during which 0.44 in. of rain fell, this being 8.5 per cent of the total of the storm. During 10 min. 0.77 in. fell, or 15 per cent of the total. In 30 min. a fall of 1.64 was recorded, or 32 per cent of the total. The fall of rain approximately in 11 hours from 7 a. m. to 6 p. m. varied between 3.61 in the Upper East River section to 7.67 in the vicinity of Staten Island. The total amount of water which ran off to the harbor according to these calculations amounted to 43,052 million gallons. A large amount of rain fell directly upon the water surfaces and this increased the total precipitation which reached the harbor to 64,062 million gallons, or 8565 million cubic feet. This is equivalent to 216,200 sec.-ft. for 11 hours.

A table, comparing the volume of storm water discharged on Oct. 1, 1913, with the volumes of water already in the harbor and passing through the harbor under ordinary weather and tidal conditions, shows the amount of rain which fell directly upon the water surfaces of the harbor above the Narrows and ran off from the land immediately tributary to the harbor was equivalent to 216,200 sec.-ft. This is nine times the average land water discharge rate of the entire Hudson River; seventeen times the dry-weather land water discharge rate of that stream and seven and one-half times the resultant rate of ebb flow of harbor water through the Narrows.

The total volume of 8565 million cubic feet is equal to four times the total mean daily land water discharge volume of the Hudson River, seven and one-half times the mean daily land water discharge of that stream and six and one-half times the total resultant ebb flow of harbor water through the Narrows in one tidal cycle of twelve lunar hours.

Comparing the volumes of storm water entering the various divisions of the harbor contained in those divisions at mean low tide, in the tidal prism and in the resultant flow toward Sandy Hook, a number of facts become apparent. The quantity of rain water which entered the Harlem River was equal to the tidal prism of that strait or the volume of water between the levels of high and low tide. Approximately the same is true for the Lower East River. The rain water which entered Newark Bay was

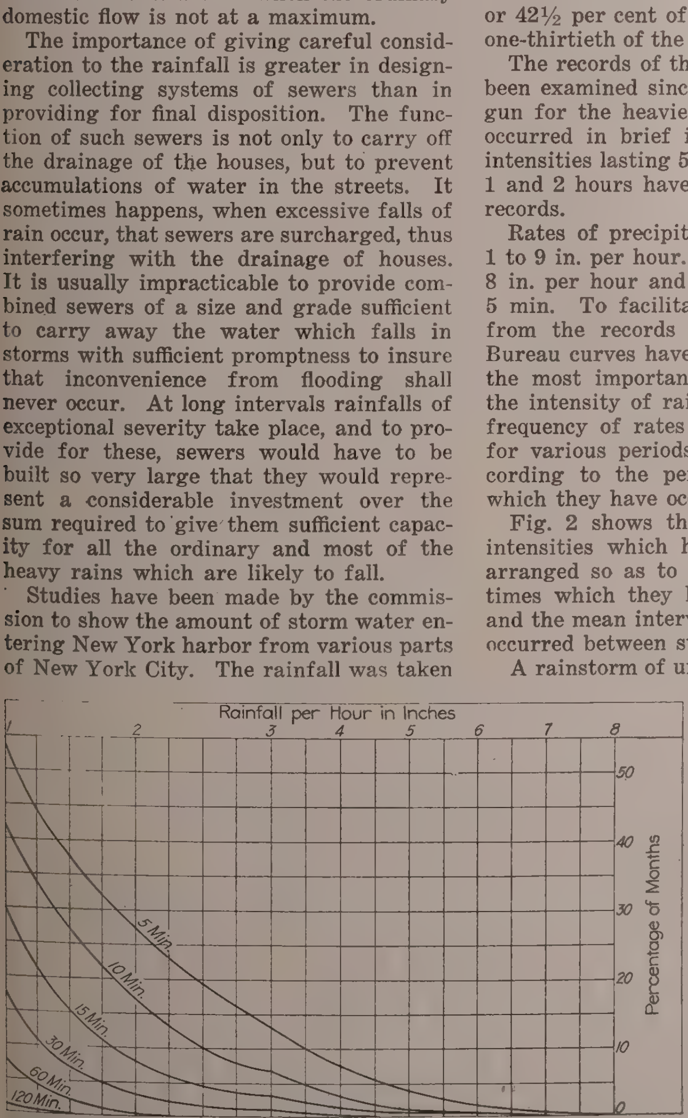


Fig. 1—Frequency of Rainfalls of Different Intensities in Manhattan

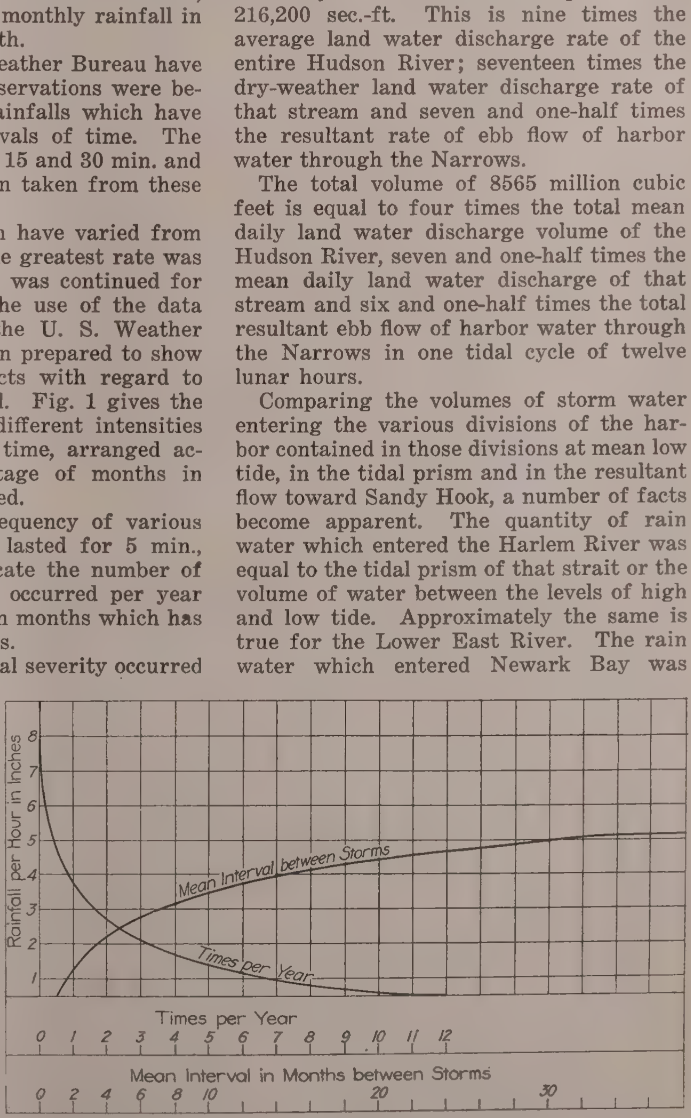


Fig. 2—Rainfalls of Different Intensities for Five-Minute Periods

about twice the tidal prism. Nearly five times as much rain water entered the harbor as the net seaward discharge under ordinary circumstances. Speaking generally, the rain water was almost equal to the tidal prism and was equivalent to about one-sixth of the total water present beneath the level of mean low water.

SEWAGE TREATMENT

It has seemed desirable to calculate the quantities of sewage materials which would be discharged into the various sections of the harbor in case the sewage was first passed through works for the removal of impurities. The processes of treatment most worthy of consideration are well established by experience. Screens are considered because of their compactness and ability to operate with varying quantities of sewage and small head; settling basins, because of their almost universal employment in sewage purification works and their efficiency in removing suspended matter; chemical precipitation, because it is one of the most efficient means for removing both suspended and dissolved matter at one process, and sprinkling filters, since they represent the most effective means of oxidizing the sewage impurities in a given area of land. It would be feasible to combine two or more of these processes in one plant.

The least effective process worth considering for the sewage discharged into the harbor is screening, and the highest degree of purification practicable is screening and rapid settlement. More purification than this requires a greater amount of land than is procurable except at great cost and would involve probable nuisance from odors and flies.

Estimates of the quantities of sewage discharged into New York Harbor show that all sections do not receive an equal share of pollution; the analyses of the water prove that the circulation of the tide is insufficient to distribute satisfactorily the excessive burden in some sections. The region of greatest pollution is close to the most densely settled part of New York City. The Lower East River and Harlem receive large quantities of sewage from both shores.

INFLUENCE OF TIDE

Another table was prepared partly from tidal data computed by the commission and partly from information supplied by the U. S. Coast and Geodetic Survey in 1909 as a result of studies made in response to the commission's request. From that table it was seen that the upper bay contains more water at low tide, has a larger tidal prism and has a larger net discharge of water than has any other part of the harbor. The Hudson River is a close second in volume at low tide, but it has a smaller prism and net flow than the upper bay. The tidal prism and volume of water at low tide are nearly the same in Jamaica Bay and Newark Bay, from which it appears that these bodies of water are almost half renewed at each tide.

In the Lower East River, the tidal prism is one-sixth the volume of water which lies beneath the level of mean low tide and the net ebb flow is about one twenty-seventh of it. Nowhere else in the metropolitan district is the net ebb flow so small in comparison with the tidal prism or volume of water at low tide. Large as is the volume of water in this division, it is evident that there is not a great deal of new water passing through it. In the Harlem River also the tidal prism is large and the net ebb flow small compared with the volume of water at mean low tide. From these figures it appears that such refreshing action as the Lower East River and Harlem River receive is due to diffusion with cleaner water at the two ends of these streams and that little renewal occurs by actual displacement with water from a neighboring section. Large though the Lower East River seems to be and swift as are its currents, it has only one-tenth as much net ebb flow as the Hudson.

RATIO OF SEWAGE TO WATER

The ratio of sewage to water in the Lower East River at mean low tide which was 1 to 244 in 1910 would be 1 to 132 by 1940. A notably low ratio is that of the Kill van Kull, which was 1 to 1470 in 1910 and would be 1 to 460 by 1940. This body of water received much of its pollution from neighboring bodies of water. The pollution of Newark Bay will increase by direct contributions of sewage until the ratio which was 1 to 1680 in 1910 will be 1 to 740 by 1940 unless measures are taken to keep the sewage out of it.

Remarkably small ratios exist between the sewage and tidal prism in most of the divisions of the harbor. The smallest occurs in the Harlem, 1 to 21.4, although the Lower East River, 1 to 32.3, is very low for the year 1910. In 1940 the tidal prism will be only eight and one-half times the volume of sewage discharged directly into the Harlem and in the Lower East River only seventeen and one-half times the quantity of sewage received.

Unlike the ratio of sewage to water below mean low tide, the relation of sewage to the tidal prism is comparatively large in Newark Bay and will be considerable in 1940. This division of the harbor, which was comparable with the Lower East River resembles the Hudson River, where the pollution is, and probably will remain, large. By 1940 there will be more sewage discharged into the Harlem than there will be tidal water passing through that stream. There will be about three times as much tidal water as sewage passing out of the Lower East River. The Hudson River and Upper New York Bay alone seem to be comparatively well supplied with water available for flushing purposes.

SCREENING AND SEDIMENTATION

Weights of sewage ingredients which will be discharged into the harbor in 1910 and 1940, assuming that the sewage is first passed through certain forms of treatment with the object of removing impurities, were computed. The efficiency of the treatment employed has been assumed as follows: The screens remove 15 per cent of suspended matter and 10 per cent of organic matter; sedimentation 60 per cent of suspended matter and 30 per cent of organic matter; chemical precipitation 85 per cent of suspended matter and 50 per cent of organic matter; and sprinkling filters 90 per cent of suspended matter and 70 per cent of organic matter.

It will be observed that the use of any of these processes would be beneficial, but that the residue to be discharged after screening or sedimentation would still leave very large quantities of polluting matter to go into the water. Chemical precipitation and the use of sprinkling filters would probably furnish all the relief needed for as many years as can now be anticipated. These latter processes are not applicable unless the sewage is carried to some point far removed from its present source for treatment.

Variations in Results with Standard Cement Sieves

Air Separation Suggested as Best Means of Measuring Fineness

TESTS have recently been carried out by the U. S. Bureau of Standards to see what order of discrepancy may be expected in fineness determinations of cement when made by the standard routine method of sieving. The conclusions, as summarized by Rudolph J. Wig and J. C. Pearson in Technologic Paper 29, are given below.

Employing the present standard method of sieving, the greatest attainable accuracy in single fineness determinations of normal Portland cement on a standard No. 200 sieve—that is, the greatest attainable accuracy in checking uniformity of samples—is about 0.2 per cent.

"Standard" No. 200 sieves may differ in their sieving values by considerable amounts, such that their corrections to the ideal No. 200 sieve may be at least as great as 0.7 per cent. Errors of at least 0.5 per cent may be looked for in single fineness determinations of normal cements on a standard No. 200 sieve when made in the usual routine manner.

Deviations exist in the sieving values of "standard" No. 100 sieves of a magnitude, roughly, one-half the corresponding values for No. 200 sieves as given above. Personal equation appears to be appreciable in hand sieving, as in most laboratory operations, the observed values being as great as 0.3 per cent.

The rating of a sieve by some system of demerits assigned from direct measurements appears to be an interesting possibility, and worthy of further study. Should a system be worked out to give reliable indications, say within 0.2 per cent or 0.3 per cent of the observed sieving value of a sieve, it will add greatly to the value of the certificate now furnished with standard sieves.

AIR SEPARATION PREFERABLE

It seems evident that both sieving tests and the interpretation of measurements on sieves are subject to considerable discrepancies, and the question arises as to whether some other, more reliable, method of determining fineness cannot be made available. The sieve at best is a measure of the coarseness of finely ground material rather than the fineness, and experiments now in progress at the Bureau of Standards indicate that air separation will offer a more satisfactory means of determining fineness than mechanical sieving.

In conclusion it may be stated that a tolerance of 1 per cent from the specification should be allowed with the No. 200 sieve and 0.5 per cent from the specification with the No. 100 sieve, every care being taken to conduct the test in strict accordance with standard methods. These tolerances should be considered as minimum values, since they are based upon the results obtained by careful and experienced observers; therefore it should be emphasized that greater differences are possible in ordinary routine testing.

Quicksand Excavation at Battle Creek

Paper Read by W. W. Brigden, Engineer Board of Public Works, at the Annual Meeting of the Michigan Engineering Society, January 13 to 15

IN constructing a pumping station for an artesian well-water supply, with a basin underneath holding about 50,000 gal., and laying sewer pipe in trenches 12 to 18 ft. deep to carry the water by gravity into the basin, the most difficult piece of quicksand work ever undertaken by the writer was encountered. Attempts to overcome the difficulties brought about by quicksand are generally met by more than ordinary care in sheeting the trenches, by using U-shaped guards to surround the pipe while being laid and by cradles of wood, gravel, piles and oakum, cloth rolls and other special contrivances for keeping the cement in the joints until well set. Cloth rolls, etc., should go all the way around the joint, not the lower half only, as otherwise the ground water will rise and wash the cement out of the joints before it is set. The main departure from ordinary practice in the work undertaken was the driving of small wells 2 in. in diameter along one side of the trench, connecting them by means of 2 to 4-in. pipes to pumps, and pumping day and night, using traction engines for power. Wells on both sides would probably be better in most cases.

Trenches

The trenches to be excavated were from 3 to 4 ft. wide and from 12 to 18 ft. deep. The pipe laid was 12 in. and 15 in. in diameter. Ordinary 2 x 10-in. yellow pine sheeting 14 ft. long was used, the rangers being 3 x 8 in. and 14 ft. long. In those cases where the trench was too deep for this length of sheeting or the quicksand was more than ordinarily difficult to handle, Wakefield sheet piling made of three pieces of 1-in. lumber was also used. It was necessary most of the time to mow large amounts of swamp grass to stop up cracks between the planks, and at times cloth cement bags filled with sand were utilized.

Quicksand was found from 1 to 3 ft. below the surface, and in most cases extended nearly to the bottom of the trench. There were innumerable boulders where the bottom of the trench came to or near the Marshall sandstone rock, which was the case along the major portion of the trenches. In some cases these boulders were so plentiful as to form a continuous line for 30 to 40 ft., with only 1 or 2 in. between the stones, and again were so large as to reach entirely across the trench and jut out on both sides to a distance of 1 ft. or more. No other way was found to dispose of the largest boulders except to drill holes in the top and blast them with dynamite. This process was objectionable, as it almost invariably nearly wrecked the adjacent sheeting. When the boulders were small enough to be taken up by a derrick they were removed by that means.

When the excavation reached the sandstone rock, conditions for pipe laying were generally much better than where the pipe was laid in quicksand, as in the latter case the quicksand often came in at the sides and also up through the bottom of the trench, displacing a pipe when it was almost laid. The conditions were also comparatively good when the pipe was laid in the till which overlays the sandstone rock. This till contained most of the boulders. During the process of excavating one of the trenches where the sandstone rock was not reached, no pipe was laid for three or four days, although constant attempts were made. It was while working on this trench that the ordinary system of trenching and pipe laying was abandoned and 2-in. wells were put down 10 to 20 ft. apart reaching below the level of the trench bottom, to pump the water out of the quicksand. At times this was so successful as to enable the workmen to get within 1 or 2 ft. of the bottom of the trench without any serious trouble from water, and in one instance, where the material was a very small-grained quicksand, the sewer pipe was laid in the dry trench for about 40 ft. In all other instances, although the 2-in. wells were driven below the bottom of the trench, there still remained some free water, which brought in quicksand and interfered in a measure with the pipe laying. But most of the water was removed by the pumps, and without their aid some of the pipe could not have been laid.

Wells to Remove Water in Sand

While the driving of wells to remove water in quicksand is not entirely new, and in fact has been used a number of times, it is still a rare way of doing such work, and so far as the writer is aware has not been undertaken in this vicinity for the purpose of laying sewer pipe. While sewer pipe was used in these trenches, it was for the purpose of carrying water from 5, 6 and 8-in. artesian wells to the basin under the pumping station, but the methods apply equally well to sewers, the only difference being that the pipes used by us were laid on a level grade. Water is pumped from the basin into the force main. This method of getting water to the basin is enormously expensive, as much of the work cost $15 per linear foot of trench; but it has many advantages, especially with the centrifugal type of pump that is to be used in our pumping station. All the sand, pieces of rock, and other material coming from the wells, pipes and manholes are deposited in the basin and removed at convenient times by hand, so as not to abraid the working parts of the pumps. There is also opportunity for aërating the water for iron removal by weirs across the waterway in the basin, a little space for storage is provided and suction is not so easily lost as in direct pumping from wells.

With the experience gained in putting down about 1000 ft. of trench it was discovered that several errors had been made. The well contractor supposed it was only necessary, where the bottom was in quicksand, to drive the well points 1 ft. or so below the bottom of the trench, and he drove the first three of four wells in that way. The writer objected to this mode of procedure and insisted that the wells be driven until the whole point, about 3½ ft. long, was below the bottom of the trench, and the wells were then driven deeper. It was discovered later that even this was not deep enough and that a well point should probably be driven so that its lower end would be 4 or 5 ft. below the trench. The line of wells was laid out on one side of the trench so as not to interfere with the digging, which was done on the other side, and the wells were placed 4 or 5 ft. outside the sheeting. This was found to be too close to the trench, for when the ditch was dug nearly to its bottom the quicksand between the well point and the bottom of the trench became the line of least resistance and air passed through from the trench to the point, thus greatly lessening the amount of water that could be removed, and on a few occasions letting in so much air that the water could not be raised the necessary 15 ft. until the pumps were placed in the very best condition or some of the poorer wells disconnected from the system. A few of the well points or screens were stripped by striking boulders. Some of these were removed at the time of driving, but in the case of others the injury was not discovered, and these wells, of course, were in poor condition to screen very fine sand and considerable quantities passed through with the water. The sand at times filled the pipes from the wells to such an extent as to stop pumping.

It was supposed when adopting the 2-in. well system that it would be necessary to purchase some unusually fine screens for wells where not driven into the sandstone rock, but this was found unnecessary. The finest quicksand passed through could probably have been handled well with the ordinary No. 60 gage screen, although No. 80 was used for that part of the work. It was necessary to send to Chicago for the No. 80 screens, as they could not be obtained in Battle Creek. The wells in those trenches where the bottom was in or close to the sandstone rock were driven without screens and to a depth that would shut out the quicksand, this generally being from 4 to 7 ft. into the rock.

Trouble from Ground Water

Although not more than from 100 to 300 ft. of trench were opened at any one time, the volume of water coming into the trenches, leaking through joints and into manholes and the basin was so great as to require constant pumping during the latter part of the construction, amounting to from 500,000 to nearly 1,000,000 gal. per day. Even when a 6-in. centrifugal pump was attached to one of the 8-in. wells and pumped to its full capacity—2,000,000 gal. per day—the ground water in the immediate vicinity was not lowered more than 5 or 6 ft. and the difficulties of running the pump became so great, owing to running off of the belt, losing suction, priming, etc., that the process was abandoned and the 8-in. well pipe carried up high enough to prevent any overflow into the trench.

The 6-in. and 8-in. wells flowed continuously near the level of the ground, and, as valves were required on each one and the lines of sewer pipe to the basin were 12 to 18 ft. deep, it was necessary to cut off the well pipes at that depth and build a manhole at each to make the valves accesssible for repairs, renewals, etc. The flow from a well at the depths stated was at a rate of 600 to 1000 gal. per minute and if it ran even for a few minutes would have ruined the trench and piping in the vicinity. At first there appeared to be no practical way out of the difficulty, but later it was solved by the use of a plumber's test plug. The plug was taken apart and a long handle or stem put in place of the short one ordinarily used. By this means two men could shove the plug down the pipe, turn the nut that expands the rubber disc and when it filled the well pipe the men could unscrew a section and screw on the valve. In most cases it was necessary to cut off the pipe with a hacksaw; then with a short piece of pipe and a sleeve put on by means of a lead joint the valve was placed near the bottom of the manhole. By screwing 10 or 15 ft. of pipe into the valve, reaching the surface, the plug could be withdrawn and the valve closed, after which the pipe above the valve was removed.

The rising of the ground water after the sewer pipe was laid and filling began sometimes occurred so soon after laying pipe that the cement did not have time to set. The water then forced out a small section of cement from the joint, leaving a place for water to leak into the pipe to the basin. To repair these leaks was a difficult matter. The first one was repaired by again digging the trench and sheeting as before, but as this cost about $125 for two small leaks near each other a cheaper plan was sought. The well contractor was called upon to drive a pipe 4 to 6 in. in diameter to the point where the leak was found. The location was determined by means of a float with candles stuck into it, the float being pushed in from the manhole by sewer rods or short lengths of ¾-in. pipe screwed together. The depth was fixed by use of the level. Care must be exercised to do no injury to the sewer pipe when driving the wrought-iron pipe when nearly down to the leaking joint. Within ten or fifteen minutes after the pipe was driven and the sand pumped out of it the leak was stopped by means of loose concrete and concrete in small cloth packages rammed down the pipe.

PUMPING

Water has been pumped from five of the 6 and 8-in. wells now in use to the extent of a 5,000,000-gal. rate per day for several hours and there were still 5 or 6 ft. of water in the basin above the bottom of the suction pipes. An estimate of the amount of water to be obtained from these wells with the present equipment gives a rate of at least 6,000,000 gal. per day, with a strong probability that 7,000,000 to 8,000,000 gal. could be obtained. Several years ago five wells, with a traction engine and a centrifugal pump placed at each, were pumped day and night for ten days, the wells yielding 8,000,000 gal. of water per day. This, however, required drawing down the water in the wells to a depth of 5 to 7 ft. below the level of the bottom of the basin under the pumping station. This test was made during the latter part of an unusually dry summer.

So great are the hazards connected with quicksand excavations that under no circumstances would the writer recommend letting a similar work by contract, either on a lump-sum bid or a price by the foot, for any attempt to make a fair proposal would be a worse risk than bucking a faro bank. If the party undertaking the work can command the needful outfit of tools and machinery, which should include light hoisting machines of some kind, it might be well to undertake it, as was done in this case, by day's work, but if done by a contractor it should be on a percentage basis or in some other manner which would eliminate the unjustifiable risk he would be obliged to take. All contracts for sewers in wet materials or in earth and rock, the nature of which has not been carefully investigated, is mere gambling and its natural outcome is large losses to the contractor because of unlooked-for troubles.

Construction of Ohio River Lock and Dam 12

Large Concrete Structures on Pile Foundations Built in Cofferdams Repeatedly Submerged by Floods and Battered by Driftwood and Ice

LOCK and Dam 12 of the Ohio River improvement below Pittsburgh are low concrete structures built by the U. S. Government to improve the navigation of the river. The river bed consists of sand and gravel about 20 ft. deep, overlying approximately level shale rock, and the river has a low-water depth of 2 to 9 ft., with sloping banks from 50 to 60 ft. in height. The construction involves a large amount of pile-driving, concreting and pumping and has been carried on in sheet-pile and gravel cofferdams, high enough to exclude ordinary stages of water. They have been subjected to very high floods and have withstood repeated submersion. They have also successfully resisted impact and pressure from large amounts of drift and ice and have enabled the site to be pumped dry except in high floods.

Lock Wall and Main Cofferdam

GENERAL DESIGN

The dam crosses the river at Martin's Ferry nearly at right angles, and with the locks forms a continuous concrete structure, T-shape in plan. A power house is built at the intersection of the lock and dam, and both lock and dam masonry rests on foundation piles driven to rock. The dam is 1120 ft. long, including a 700-ft. navigable pass, two 91-ft. bear-trap sections, three bear-trap weirs, and a 200-ft. weir with end abutment on shore. The pass has a concrete bottom slab 35 ft. wide, 5 ft. deep and 3 ft. in the clear below low-water level. It serves as a floor on which are installed Chanoine wickets 11 ft. high.

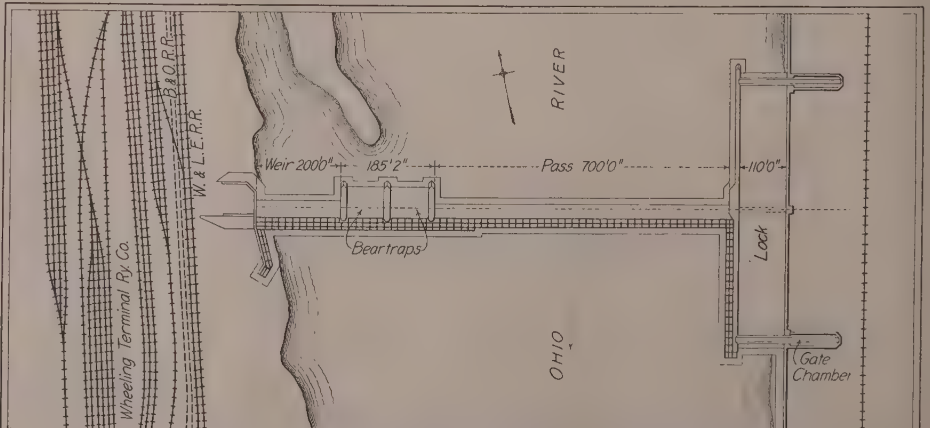
The upstream face of the full length of the dam is protected by a line of 10-in. sheet piles driven to rock, with riprap outside. The downstream face of the dam is protected by a continuous line of rock-filled timber cribs, 20 ft. wide, which extend several feet below the original surface of the river bed and have a solid deck of 10-in. hardwood timber, beyond which is a further protection of heavy riprap extending from the top to the bottom of the crib and with the upper surface sloped downstream to the river bottom.

The 110 x 600-ft. lock is at right angles to the dam and is adjacent and parallel to the east bank of the river. The river wall is about 23½ ft. high and the shore wall about 25½ ft. high. The steel gates slide horizontally at right angles to the locks into concrete chambers constructed in the river banks. The face of the river wall is protected by rock-filled cribs and riprap, and the shore wall retains a backfill supporting the concrete pavement of an esplanade.

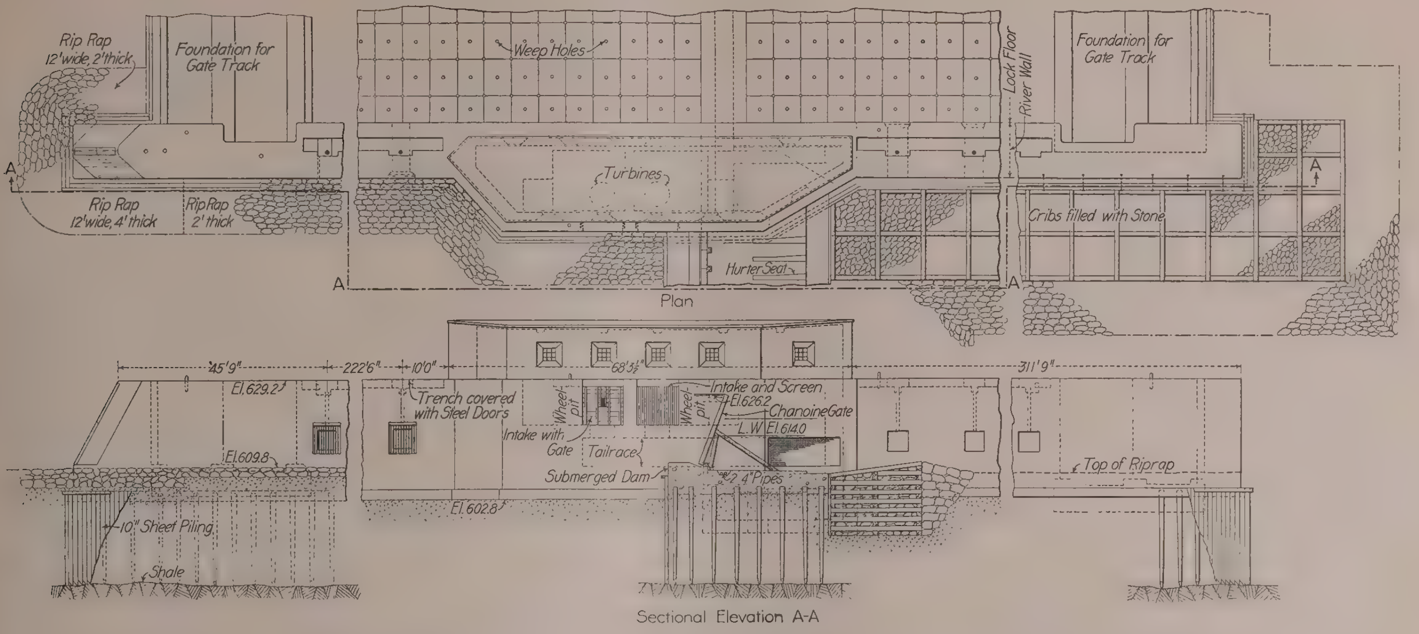
The floor of the lock is paved with 6 x 8-ft. concrete blocks 18 in. thick, cast in place. The shore wall of the lock is extended about 600 ft. upstream and downstream, from the gates to form guide walls, on pile foundations. The upper guide wall is 11 ft. wide at the base and 5 ft. wide at the top, with a height of 24.4 ft. and top of wall 5 ft. above upper pool level. The lower guide wall is 9 ft. 6 in. wide at the base and 5 ft. wide at the top, with a vertical height of 21 ft. The top of the wall is 6 ft. above lower pool level.

COFFERDAMS

As the contract required 350 ft. of the navigable pass included in the lock cofferdam to be completed and its cofferdam to be removed before commencing the bear-trap foundations, it was necessary to construct the work in two large cofferdams, which successively closed the east and west sides of the channel, always leaving the other side open for navigation. The cofferdams for the guide walls are similar in construction to those used for the lock, but are only 10 ft. thick. The cofferdams inclosing the lock and dam are of the box type and have two rows of wooden sheet piling spaced 16 ft. apart on the average. The



Location Plan of Lock, Navigable Pass, Dam and Weirs



Plan and Elevation of Lock, with Section through Dam and Movable Gate

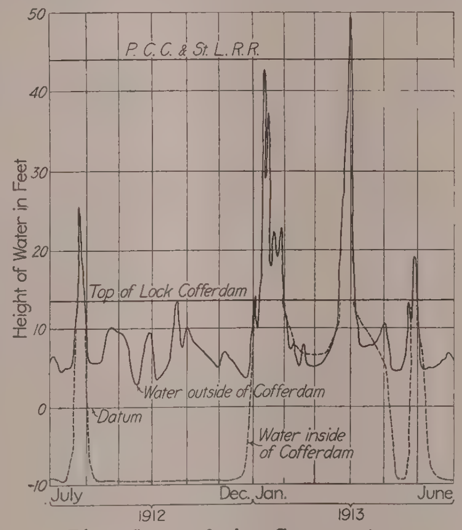
piles are driven against waling pieces on exterior round piles through which horizontal transverse tie rods are passed to resist interior thrusts. The top of the cofferdam is at least 16 ft. above bottom or low-water level, thus permitting the work to be carried on at all stages of the river up to 16 ft. gage.

The cofferdams were filled with materials dredged from the river bottom above the dam, brought down in barges, and deposited inside the sheeting by clamshell buckets operated from derrick boats. The top of the fill in the cofferdam is covered by a protecting layer of concrete. Both sides of the cofferdam are reinforced by sloping embankments of dredged materials. The stability and resistance of this construction were such that the dams passed without serious injury through a number of severe floods and have been in constant service for many months.

The contract provides for the payment for the cofferdam in three instalments—one of 50 per cent when the cofferdam is entirely pumped out ready for the execution of work in any part of it, one of 35 per cent when the inclosed permanent work is completed, and the remainder on the completion of the contract.

The cofferdams were unwatered and pumped dry by a movable pumping plant consisting of three pumps, two having a capacity of 20,000 gal. per minute, which were installed on a 32 x 80-ft. barge equipped with three boilers having a combined capacity of 270 hp. When floods were imminent closed gates were opened in the cofferdams and water admitted from the river to fill the cofferdams and prevent excessive unbalanced pressures.

The river wall of the lock was made in alternate sections separated by open spaces which were afterward filled in, completing the structure. The concrete was delivered by cars on a service track from the central plant to traveling derricks which placed it in the forms. The forms were made with full-length wooden panels secured by waling pieces, tie rods and inclined braces, and were handled by the traveling derricks.



River Stages during Construction

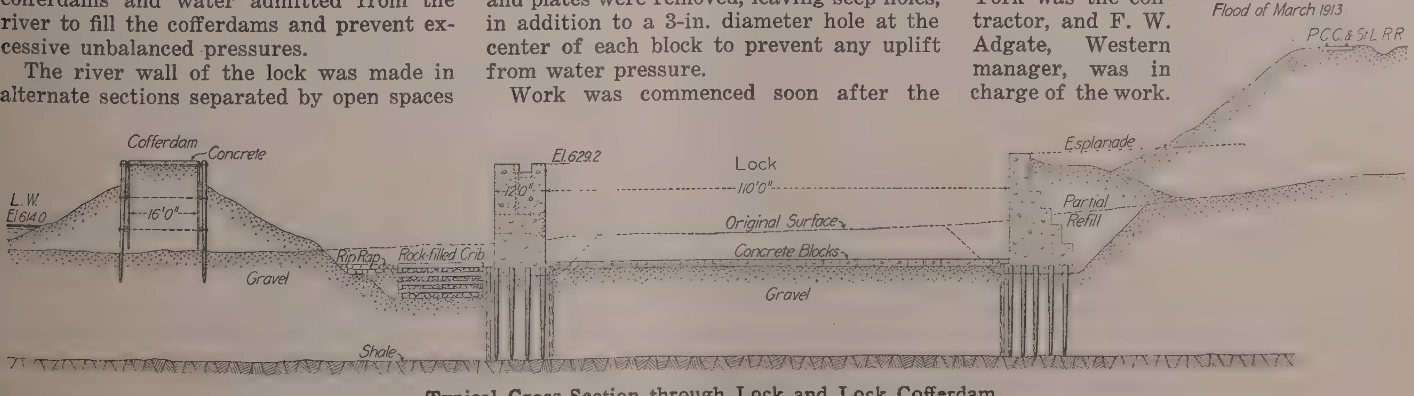
The concrete for the lock floor was deposited on the level surface of the ground in steel forms made of ¼-in. vertical steel plates, with their ends engaging slots in all four faces of 6 x 6-in. x 2-ft. corner stakes. When the concrete was partly set the stakes and plates were removed, leaving seep holes, in addition to a 3-in. diameter hole at the center of each block to prevent any uplift from water pressure.

Work was commenced soon after the award of the contract in 1910 and has been delayed by a number of high floods to which this river is subject. Those of the year commencing July 1, 1912, are indicated by the accompanying profile, which shows four of them rising above the top of the cofferdam and one of them to a height of nearly 30 ft. above the cofferdam. The dotted line shows the efficiency of the cofferdam and pumping between floods in keeping the level down to a stage permitting the work to continue between floods. During one of the floods the water rose to within a foot of the sills in the first-floor windows of the contractor's office building, but did not move the building. A considerable amount of the contractor's plant and equipment was submerged and a large quantity of mud and silt was deposited in the locks, where a force of men was necessary to construct trenches and drain it to sumps where the suction pipes from the pump boats were located.

During January, 1912, the quantity of ice coming down the river entirely covered the north end of the lock cofferdam, but did not carry it away. In March, 1913, the flood reached stage 35.7, bringing down large quantities of drift, which lodged along the river bank and cofferdam and was removed by derrick boats.

PERSONNEL

The work was designed and executed under the direction of Brig.-Gen. Dan C. Kingman, Lieut.-Col. H. C. Newcomer and Maj. F. W. Altstaetter, Corps of Engineers, U. S. A. The Foundation Company of New York was the contractor, and F. W. Adgate, Western manager, was in charge of the work.



Typical Cross-Section through Lock and Lock Cofferdam

Trucking Costs

Comparative Figures for Horses, Motor Trucks and Tractors, Taking into Account Both Fixed and Operating Charges

As the result of an extended investigation into the cost of transporting goods and material over highways, made for a client of the New England Audit Company, Walter M. Curtis, manager of the company's engineering department, reported the following general conclusions:

1. There is a great advantage in using large hauling units whenever the nature of the load makes a large tonnage practicable.

2. Fixed charges form a considerable portion of the table cost in the case of the motor truck.

3. Cost per ton-mile is reduced with increase in daily mileage.

4. Elimination of "idle time" is important.

5. Tire expense is a large direct operating charge in the case of the regulation motor truck.

Comparative costs were compiled for the use of horses, and of motor trucks and tractors with and without trailers. The records cover a contracting job, coal delivery and long-distance trucking. The figures for the contracting job, reproduced in the accompanying table, are for a large hauling contract, in which special dumping bodies were used, loaded from chutes. The haul was 2 miles.

For the horse-drawn equipment the fixed charges were based on interest at 5 per cent, insurance (employer's liability, fire and liability) at Massachusetts rates outside of Boston, depreciation of 12 per cent on horses and 10 per cent on harness, wages of drivers, and feed, stable and veterinary charges of $5 per week per horse. Maintenance charges were estimated at 1 cent per mile per horse for shoeing and 1 cent per mile for wagons and harness. Depreciation on wagons was based on a total mileage of 50,000.

Fixed Charges

Interest for the motor equipment was figured at 5 per cent on truck value minus tires, and insurance was figured at Massachusetts rates outside of Boston. The other fixed charges embraced drivers at $3.50 per day, and garage charges. Maintenance was figured at 3½ cents for the 10-ton tractor and 4 cents for the 6-ton truck. Depreciation was based on 100,000 miles total mileage and on truck value minus tires. Gasoline was estimated at 20 cents per gallon, considering that the 6-ton truck would go 3 miles and the 10-ton tractor 2¾ miles on a gallon. The figures for oil and grease were taken at about 20 per cent of those for gasoline. The charges for tires were based on 8000-mile guarantees, the price being according to the size of the truck.

It is pointed out by Mr. Curtis that the costs are based on 300 working days per year and that the cost per ton-mile will increase materially if there is considerable idle time. The Knox-Martin tractor was the basis for the tractor figures, and the report is presented in booklet form by the Knox Automobile Company, of Springfield, Mass.

Comparative Costs on Contract Work for Horses, Trucks and Tractors

	A	B	C	D	E
Investment	$9,600	$6,000	$4,600	$6,600	$5,500
Annual Charges					
Interest	$480	$273	$210	$303	$256
Insurance	290	241	221	250	234
Depreciation, horses	672	....	....	....	....
Drivers	6,000	1,092	1,092	1,092	1,092
Stable or garage	4,160	300	300	300	300
Total fixed charges	$11,602	$1,906	$1,823	$1,945	$1,882
Maintenance, horses	768	....	....	....	....
Maintenance, trucks or tractors	....	576	432	480	420
Maintenance, wagons or trailers	384	....	....	120	120
Depreciation, trucks or tractors	....	785	482	726	402
Depreciation, wagons or trailers	384	....	245	144	420
Gasoline	....	970	1,047	873	960
Oil and grease	....	152	209	175	192
Tires	....	960	720	825	600
Total operating charges	$1,536	$3,463	$3,135	$3,343	$3,114
Total annual charges	$13,138	$5,369	$4,958	$5,288	$4,996
Miscellaneous					
Total earth hauled in cubic yards	24,000	12,600	28,800	21,000	48,000
Ton-miles	64,800	34,020	77,760	56,700	129,600
Cost per cubic yard in cents	54¾	42½	17¼	25¼	10½
Cost per ton-mile in cents	20¼	15¾	6½	9¼	4

A, 8 2-horse teams, 4 trips each per day; investment—16 horses at $350, $5,600; 8 wagons at $500, $4,000—total, $9,600.

B, 1 6-ton truck with 3½-yd. body, 12 trips per day; investment, $6,000.

C, 1 10-ton tractor with 8-yd. body, 12 trips per day; investment—tractor, $3,750; wagon, $850—total, $4,600.

D, 1 6-ton truck and 1 trailer with 3½-yd. bodies, 10 trips per day; investment, $6,600.

E, 1 10-ton tractor and 1 trailer with 8-yd. bodies, 10 trips per day; investment—tractor, $3,750; wagon, $850, trailer, $900—total, $5,500.

Fertilizer from Sludge

Fermentation Process with Brewery Waste at Dublin Sewage Works Effects Recovery of Nitrogen

Sludge from the sewage-outfall works at Dublin, Ireland, is used as a base in the manufacture of fertilizer by the Dickson patented process, in which brewery waste is introduced into the sludge to promote rapid fermentation. The workings of the process were studied by John D. Watson, engineer to the Birmingham, Tame and Rea District Drainage Board, and his estimate of its value is summed up in the following sentences from his presidential address, delivered on Jan. 7, to the Institute of Sanitary Engineers: "Its merits are sufficiently conspicuous to warrant me in saying that its discovery marks a decided advance in the treatment of sewage sludge which no engineer can afford to ignore. Apart from the hygienic and profit-making elements, I attach much importance to the utilization of the nitrogen of sludge, which, up till now, has been literally thrown away."

Process

The process, according to Mr. Watson, may be divided into two main sections: (1) The fermenting of the crude sludge mixed with a small percentage of yeast, and the separation of as much water as possible by this means, and (2) drying the resulting sludge, mixed in this case with certain compounds containing phosphates and potash to produce a fertilizer.

The sludge produced from the fermentation has a water content of about 82 to 84 per cent. Thus in the case of a 95 per cent crude sludge, about two-thirds of the water is separated in the fermenting process, and the remaining one-third in the dryer. It is found that this density figure is obtained whatever the quality of the crude sewage, and therefore the variation in the amount of water to be separated per pound of solid matter due to variation in density of crude sludge is entirely taken charge of by the fermenting process, and the work of the dryer remains practically constant per pound of solid matter.

Addition of Yeast

Sludge is pumped into an overhead tank and from there a pipe is taken to an open trough leading to a small screen. At this point about 0.5 per cent of its weight of ordinary yeast is added to the sludge, this yeast being mixed with water to enable it to be supplied through a pipe.

The mixture of sludge and yeast is then pumped through a "heater," which consists of a number of pipes placed in the path of the hot air from the furnace, and is then delivered to the fermenting troughs, which are eight in number, each about 50 ft. long by 4 ft. wide, and holding about 3000 imp. gal. Under each trough is a hot-air duct, and the sludge is kept as far as possible at a temperature of about 90 deg. Fahr. during the fermenting process. In about twenty-four hours it is found that, as the result of the fermentation, there is a distinct separation of water, the sludge at a density of about 83 per cent occupying the surface, while the water can be readily drained away from underneath.

Before being taken to the dryer the fermented sludge is mixed with a compound of phosphates and potash in about equal proportions by weight of sludge and compound based on a dry, solid matter, because, although a satisfactory manure can be made from the sludge only, it is found that a greatly enhanced value is given to the final product at a much less than proportionate cost.

Drying Machine

The mixture, containing about 73 per cent water, is then pumped into the dryer, which is one of the most important pieces of apparatus in the whole plant. It consists essentially of a cylindrical vertical casing containing a series of arms and platforms revolving upon a center shaft, and between fixed arms and platforms. The platforms have large perforations in the shape of sectors of a circle, and the mixture which is fed in at the top is scraped over the surface of the plates and gradually falls through the dryer to the outlet at the bottom. Air at a temperature of about 450 deg. Fahr. is blown in at the bottom and passes out at the top of the machine. The dried product then falls into a disintegrator, consisting of a revolving paddle, which beats up the product into a powder, which is blown out at one end of the machine by a draft of hot air.

The process is spoken of throughout as a fermentation one, but it is probably even more correct to speak of it as a digestive process. The introduction of yeast into the sludge evidently provides stimulating food for the putrefactive bacteria, as the rapidity with which the solid part of the sludge is separated from the liquid part is one of the extraordinary features of the process, according to Mr. Watson.

Driving and Lining Point Defiance Tunnel

Poling Boards and Semicircular Shields Were Used in Excavating Bore of Northern Pacific Railway near Tacoma

TO avoid steep grades on the Tacoma-Tenino line of the Northern Pacific Railway near Point Defiance Park, just within the northern limits of the city of Tacoma, Wash., a tunnel 4391 ft. long is being driven. The first 1600 ft. on the east end are on a 1-deg. curve to the left; this is followed by a 1350-ft. tangent, and then for the remaining length the center line swings again to the left on a 3-deg. curve. The bore was made for a double-track line, the inside clear dimensions being 28 ft. in width and 27 ft. in height. The arch is a semicircle with a 14-ft. radius, except at one point, where heavy ground caused some settlement in timbering and made it desirable to use a special five-centered arch, dropping the crown in order to get the required clearance over the tracks.

EXCAVATION METHODS

The material traversed by the tunnel is a very compact sand with occasional clay pockets and some streaks of coarse sand and gravel mixed. All of the material carried enough moisture to make it compact, and it was found that when the sand was loosened and left to take its own slope it would stand on about a 1 to 1 slope as long as the moisture remained in it. No excessive quantities of water were encountered; occasionally there was enough to cause a decided drip, but nowhere a sufficient amount to cause any great inconvenience.

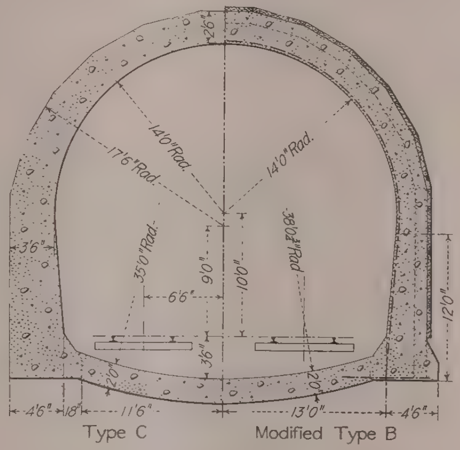
Excavation for the easterly approach involved the moving of about 106,000 cu. yd. of material, while on the westerly approach about 17,000 cu. yd. of material had to be removed. These portal approaches were taken out with steam shovels. Excavating on the east portal was begun on April 6, 1912, and continued until Nov. 12, 1912, when sufficient excavation had been done to allow work to be started on the wall plate drifts and arch of the tunnel proper. Excavation for approach cuts was then stopped during the remainder of the rainy season. The ordinary poling-board method was used in excavating wall-plate drifts on both ends, and this work was continued on the east end until Jan. 19, 1913. On this date a semicircular shield, 15 ft. long, designed to run on shoes on the wall plate, was installed.

Excavation of the portal approaches on the west end was begun on Feb. 20, 1912, and completed on April 19, 1912, on which date work was begun on driving the wall-plate drifts and top-center drifts. On this end the poling-board method was used until Aug. 24, 1912, when the installation of a shield similar to the one in use on the east end was begun.

The first lining was built of 12 x 12-in. timbers with an eleven-segment arch, 12 x 12-in. wall plates, and posts and mud sills of the same size. Timber sets were spaced throughout the tunnel according to the character of the material encountered; some were on 4-ft. and some on 3-ft. centers, but for the majority a 2-ft. spacing between centers was adopted.

ADVANCING THE SHIELDS

The shields consisted of 1-in. boiler plate reinforced with angle irons and a frame of 18-in., 70-lb. I-beams. This shield was moved ahead by bracing twelve 50-ton hydraulic jacks against it while men removed the material from the cutting edge. The jacks were worked against the 12 x 12-in. permanent timber lining. The working pressure on the jacks at the west end averaged about 850 lb. per square inch, while on the east end the pressure was about 1350 lb. per square inch, the difference being due to the fact that the shield on the east end was about 4 ft. longer than the one used on the other end. The method of advance was to jack the shield ahead 1 ft., block up, jack ahead another foot, reblock, and so on until 4 ft. had been excavated. The two sets of timbers were then installed, the 4-in. lagging put on, and such packing as was necessary done with timber. This packing, however, was a very small item, and varied from about 1 in., the usual thickness, to 2 or 3 in.



Two Types of Tunnel Section

The first advance of the shield on the west end was begun Oct. 3, 1912, and on the east end the shield actually began work Feb. 27, 1913. Shields met June 14, 1913, the west shield having moved 2430½ ft. in 237 working days and the east shield 1038½ ft. in 101 working days—two ten-hour shifts being a working day in each case. The east shield averaged 10.28 ft. per working day, while the west shield averaged 10.25 ft. per working day.

MOTIVE POWER AND VENTILATION

The bench on the east wing was removed by driving a bottom center drift large enough to accommodate 4-cu. yd. cars and trapping into them, hauling the material above and hand-mucking that below. The material from the wall-plate drifts and the top-center drifts was handled to the front by conveyor belts driven by motors, and the bench was kept to within 25 or 30 ft. of the advancing heading. On the west end the bench was excavated by a model 40 Marion shovel loading into 4-cu. yd. cars.

All equipment used to drive the west heading, behind which the major portion of the excavation was done, was operated either by electricity or compressed air. Current was supplied by the city of Tacoma to the compressor plant located at the west portal. Compressed air was used in the shovel, while conveyor belts and ventilating fans, of which there were three or four, were driven by electric motors.

The ventilating pipe for each heading was located just below the top of the arch and consisted of a 16-in. galvanized pipe, with suction fans at the portals for drawing out the air. The compressed air used in the shovel furnished a good supply of fresh air at the front. On the east end, on account of the short distance underground, dinkey locomotives burning coke were used for handling dump cars. No particular difficulty was experienced in this heading, although had the work been continued much farther underground some other arrangements for ventilation would have been necessary.

CONCRETE LINING

The actual tunnel section excavated varied considerably, of course, according to the kind of material traversed, and several types of concrete lining were used. Two of these, designated as type C and modified type B, are shown in the accompanying illustration. A length of 2643 ft. of modified type B and 223 ft. of type C were required, the remaining 1526 ft. of the tunnel being built of other types.

The depth of the invert is to be 2 ft., but this will not be laid as a continuous concrete slab, the plans calling for a 15-ft. length of the concrete invert, after which there will be a 5-ft. space, followed by another 15-ft. length of concrete, and so on, the result being an amount of concrete equal to a continuous invert through three-fourths of the tunnel. This arrangement was used to give stiffness to the side walls, which vary in thickness from 4½ to 7 ft. These walls were considered of ample thickness to safely carry any side pressure which would ever act upon them over the 5-ft. unsupported section. Material will not be excavated from between the concrete invert slabs. Drainage will be provided along the center line of the invert by a 12-in. vitrified pipe laid directly on the invert and covered by crushed-rock ballast.

PROGRESS SUMMARY

Month	Linear feet driven This month	Linear feet driven To date
1912		
February	0	0
March	0	0
April	20	20
May	120	140
June	111	251
July	139	390
August	134	524
September	14	538
October	190	728
November	229	957
December	417	1,374
1913		
January	434	1,808
February	312	2,120
March	512	2,632
April	687	3,319
May	731	4,050
June	341	4,391
July	...	.....
August	...	.....
September	...	.....

A mixing plant was established at either end of the tunnel for supplying concrete for the lining, and this work was started in the latter part of April, 1912. The work was continued during the excavation of the tunnel, but no attempt was made to keep the concrete work up with the progress of the excavation. All the concreting, including the inverts, was to have been completed by Jan. 1.

The tunnel was designed under the supervision of W. L. Darling, chief engineer of the Northern Pacific Railway, and construction has been carried out by J. C. Breedlove, assistant engineer. The work of excavating, timbering and concreting, as well as the portal approaches, was let by contract on Feb. 10, 1912, to Nelson Bennet, of Tacoma, Wash.

Railroad Fill of 1,500,000 Cubic Yards on Swampy Ground

Controlling Lateral Displacement of Underlying Material in Placing Embankment for Grade Elevation on Central Railroad of New Jersey

EXTENSIVE improvements just completed on the Central Railroad of New Jersey involve the expenditure of $500,000 for fill to raise sufficiently 3 miles of double tracks near Jersey City to permit a large proportion of the boats in the Hackensack and Passaic Rivers to pass under the bridges without necessitating the opening of the drawspans.

The swampy ground on which the high embankment was made is so soft that the fill sunk through it to harder stratum, caused serious lateral displacement and upheaval of the mud, overthrew large telegraph poles and allowed successive tiers of retaining wall to sink many feet below the surface.

The old bridges were replaced by new ones with a maximum clearance of 25 ft. above high water, and about 3000 ft. of double tracks formerly carried on the surface of the meadows, about 7 ft. above high water, have been raised to the tops of piers 30 ft. high. Some of the former approach spans have been eliminated, and the grade elevation being carried across several streets with overhead bridges extends the embankment 5000 ft. from the river spans to present grades at both ends of the section, besides 3500 ft. between the river bridges. Only two tracks are now laid on the new alignment, but the construction is such that, except at the river bridges, it can readily be prepared to receive four tracks corresponding with the adjacent sections at both ends.

SEQUENCE OF WORK

In order to maintain the high-grade tracks in the same alignment as the original low-grade tracks, provision was made for carrying traffic during reconstruction on a temporary pile-trestle double-track bypass parallel to the old alignment and located about 150 ft. from it at the river crossing where pile-trestle approaches to the drawspans were built aggregating 3000 ft. in length. Between the river bridges the bypass tracks were laid on the surface of the meadows about 7 ft. above high-water level and as close as possible to the original location, permitting the old tracks to be abandoned and the fill to be made on their site.

Work was commenced by the construction on the east side of the Passaic River of the bypass pile trestle about 7 ft. high and 600 ft. long, on which trains of side-dump cars were at first operated to receive material for the fill delivered by scows in the river. A similar trestle 300 ft. long was built on the west side of the Passaic River for the same purpose, and from these trestles all of the material was handled for the fill for the temporary tracks between the rivers and west of the Passaic River. On the east side of the Hackensack River a bypass trestle 1600 ft. long was built and handled the material, and from it the fill was extended to the end of the section with temporary trestles built over the intersecting streets and the canal. After the river trestles were completed the drawspans were moved to temporary positions in the alignment of the bypass, to which traffic was transferred.

Material for the fill consists of earth and rock from cellar and subway excavations in New York City, which were delivered on about seventy 500-yd. deck scows and unloaded by four stiffleg derricks with 3-yd. orange-peel buckets operated by 60-ft. booms. Each derrick handled about 100 yd. of material per hour, delivering it at an elevation of about 10 ft. above the water to 12-yd. side-dump cars operated by locomotives in trains of four or five cars. Between the bridges and at both ends the ground consists of salt meadows with a surface slightly above high water, and is mud with an average depth of about 12 ft. and a maximum depth of perhaps 20 ft., overlying a stratum of clay with hardpan below it. The mud is so soft that its bearing value is very small and it is displaced laterally by any considerable weight, therefore allowance was made for the fill to settle through the mud to the hard stratum.

New and Old Bridges

THREE-STORY RETAINING WALL

In order to provide room for the tracks of the traffic bypass close to the north side of the fill, to economize fill, and protect its slope, the toe on the north side of the embankment is provided with a retaining wall made of material selected from the fill. The embankment between the rivers is 32 ft. wide on top with 1½ to 1 slopes.

Just in advance of the fill the retaining wall was built with a width of 3 ft. on top, 5 ft. at its base, and a height of 5 ft. As the fill was deposited behind it and settled, the retaining wall also settled, until its top was level with the temporary or bypass tracks, when a second wall of the same dimensions was built on it on the same line. As both walls continued to settle with the deposition of more fill material, a third wall was eventually built on top of the second wall, making finally a three-story wall about 24 ft. high, extending through the soft mud to the hard stratum below and serving as a cutoff that offered very effective resistance to the transverse displacement of the soft mud. The wall settled with great regularity, without apparent distortion or injury, and reduced the subterranean movements so much that very little upheaval of the soil beyond it was apparent, and the tracks required much less shifting than otherwise would have been the case to maintain their horizontal alignment.

On the south side of the fill, where no cutoff wall was built, the displacement of the mud was unimpeded and was so great that recently constructed expensive telegraph poles carrying 100 wires about 100 ft. from the center of the fill were moved from 50 to 100 ft. transversely and totally wrecked, some of them completely thrown down and others turned to very great angles and their wires broken. The poles were abandoned on the south side of the track and the telegraph wires replaced by a cable on poles on the north side, after which no further trouble was experienced.

The work was done under the direction of the engineering department of the Central Railroad of New Jersey, of which Joseph O. Osgood is chief engineer; A. E. Owen, principal assistant engineer; J. J. Yates, bridge engineer, and L. M. Hannaford, resident engineer. H. R. Wheeler, chief engineer, and W. D. Malcolm, superintendent of construction, were in charge for the contractor, Henry Steers, Inc., of New York.

Sanitary Survey of White River

Abstract of Paper Read by Jay A. Craven, Engineer, Indiana State Board of Health, at Meeting of Indiana Engineering Society, Jan. 23

DURING 1913 the west fork of the White River was surveyed sanitarily from Winchester to the Wabash River, a distance of 388 miles, by the Indiana State Board of Health. This branch drains one-sixth of the 1900 sq. miles of watershed of the river. Part of the work was done from a houseboat laboratory, and many physical difficulties were encountered. From Winchester to Muncie the river was traversed on foot; from the latter point to Indianapolis a rowboat was used; to Martinsville the trip was made by automobile, collecting samples at the crossings. From Martinsville down the houseboat was employed, as it had been brought up the river the previous spring. For most of the trip the water was low and many hours were spent in pushing the boat across sand bars.

The greatest obstacle was at Washington, where the Baltimore & Ohio bridge, washed out in the March flood, had been replaced by a pile trestle with the bents 10 ft. apart, 3 ft. less than the width of the boat. By interceding with the Public Service Commission the officials were induced to pass the boat over the 35-ft. trestle by a wrecking crane and a construction crane. Long 8 x 10-in. timbers were placed under the gunwales of the boat and two 8 x 10-in. cross-beams inserted under the longitudinal timbers, 5 ft. from the ends of the boat. Other 8 x 10-in. timbers were placed on top of the cabin and the top and bottom timbers lashed together. Those on top were placed over 6 x 6-in. struts laid longitudinally over the cabin and the cross-pieces bolted to the struts.

The results of the examination of the 334 river samples indicate that the river is in a much more foul condition below Indianapolis than at the previous examination in 1905. Another point was the special investigation of 445 samples from local sources, wells and springs of the various cities passed through and the education of the citizens in these towns, where explanations were freely made of the work and value of it.

Balcony of Empress Theater in Portland, Oregon

Radial Girders and Trusses Supporting Inclined Cantilever Floor

THE Empress Theater in Portland, Ore., is a steel-frame structure with a balcony floor extending 98 ft. across the full width of the building and inclined at a pitch of about 24 vertical to 31 horizontal. The radial floor girders parallel to the floor surface are comparatively long and shallow and are fulcrumed on a heavy transverse truss, beyond which their cantilever bent ends project and are connected to the fascia girders, which are segments of a circular arc.

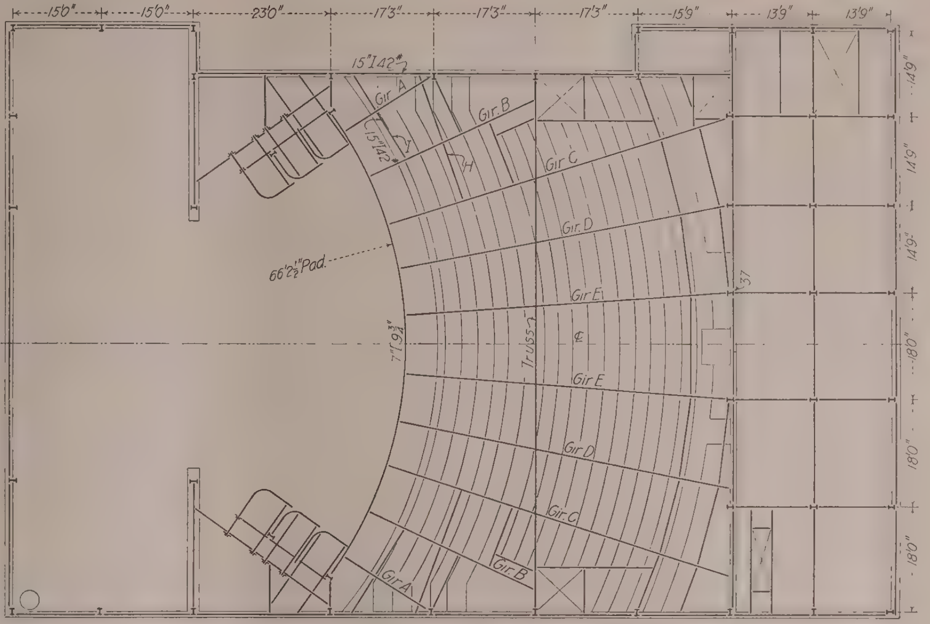
RADIAL GIRDERS

All of the main radial girders pass through the web of the fulcrum truss T and are supported by lower flange bearings on oblique seats made by bent plates with their outstanding flanges supported on the ends of vertical reinforcement angles. The top flanges also have bent-plate connections riveted to the fulcrum truss. The cantilever girders A, E and C have their rear ends connected by hitch angles to interior columns, which also support the gallery floors. The other cantilevers, B and D, are supported and anchored at the upper ends by connections to horizontal transverse girders.

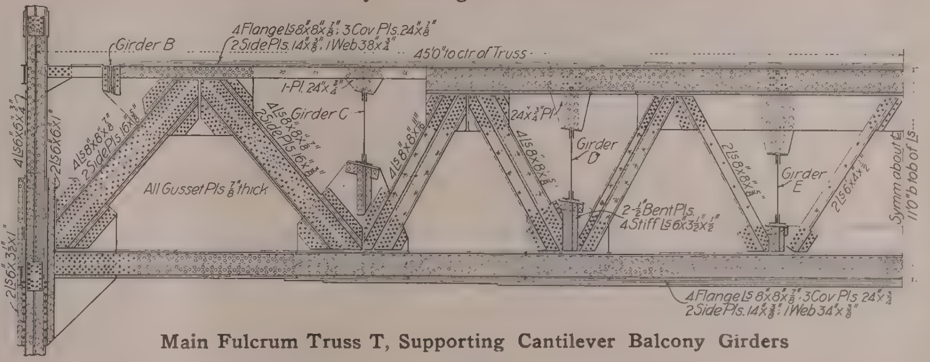
The lower ends of the cantilevers are sharply bent upward just beyond the fulcrum truss and short horizontal arms are spliced to their extremities to connect with the fascia girders. The full-length flange angles are bent to conform to the shape of the girders, and the web is stiffened by pairs of vertical angles skewed with the inclined flanges and dividing the girder webs into oblique parallelograms.

At the sides of the auditorium short cantilever girders, B, are anchored at their upper ends to the fulcrum truss T, and are fulcrumed to horizontal I-beams, H, supported at one end on the wall girders and at the other end on main cantilever girders, C. In these cases the fulcrum supports consist of heavy yokes with vertical angles field-riveted to special sections of the girder webs projecting above their top flanges and field-riveted to wing plates or extensions of hitch angles and the webs of the transverse beams.

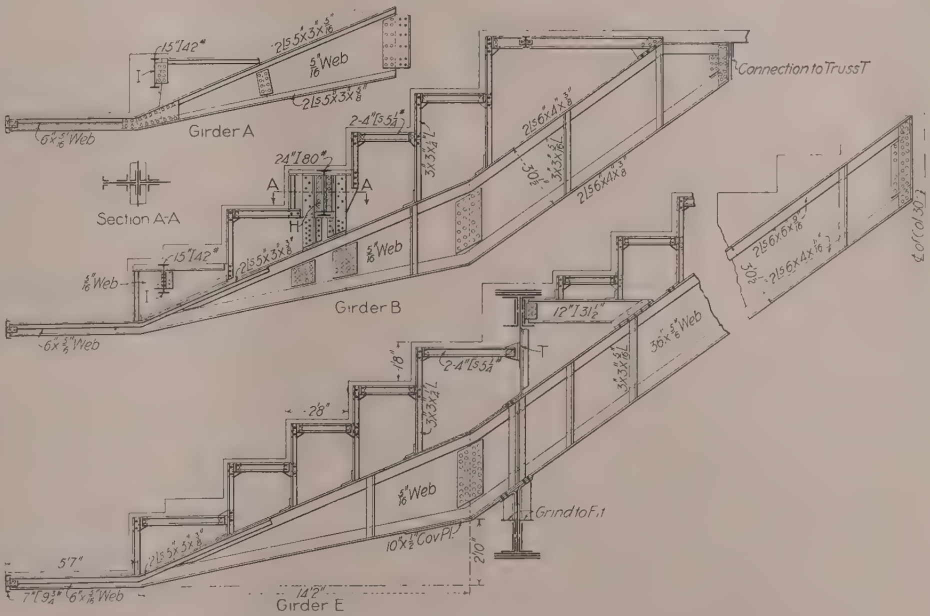
Very short cantilever girders, A, near the ends of the fascia girders resemble the extremities of the main cantilever girders and are anchored at their upper ends directly to the wall columns. Special center sections of the web plates project above the top flanges and engage hitch angles, field-riveted to the webs of short transverse beams, I, above the cantilevers, that are carried at one end by the wall girders and at the opposite end by the balcony cantilever girders B. This rather unusual system of support provides a simple arrangement of girders for a somewhat elaborate system of reaction.



Plan of Balcony Framing and Location of Girders



Main Fulcrum Truss T, Supporting Cantilever Balcony Girders



Typical Cantilever Girders, with Solid Plate Webs, Supporting Balcony Floor

FULCRUM TRUSS

The fulcrum truss T extends across the full width of the auditorium to support the balcony and the grand foyer floor beams. it is carried by the wall columns, which have special sections of web plates extending beyond the column flanges and field-riveted between the elements of the lower chord and inclined post of the truss, thus making a stiff connection integral with both members. The lower edges of these connection plates are ground to the shape of a wedge to facilitate their entrance between chord and diagonal elements. The gusset plates at lower-chord panel points are extended high enough to receive on their upper edges inclined seats for the radial cantilever girders. Corresponding top-chord gussets are provided vertically above them for top-flange connections of the girders. The truss was assembled and fitted at the bridge shop and all members were shipped loose.

The truss weighed about 55 tons and was erected at the site on falsework, the rivets being driven by pneumatic hammers. The cantilever girders were shipped whole from the bridge shops and had a maximum weight of 8 tons. The total weight of the structural steel in the building is 610 tons.

H. Bittman, of Seattle, Wash., was the structural engineer. Kingsley & Eastman, of Seattle, were the architects; Lee De Camp was the associate architect; E. M. Davis & Company were the steel erectors, and the American Bridge Company was the fabricator.

Need for Testing Commercial Sands for Use in Concrete

Abstract of Paper Presented to Indiana Engineering Society, Jan. 23, by Prof. H. H. Scofield of Purdue University

DEFECTS in natural sands which may cause concrete failures were enumerated as those having to do with composition of the sand grain, the granulometric composition and the moisture content.

Of the first defect, hardness of the sand is an important item governing strength, and for that reason a high silica content is desirable. The grains should offer as great a resistance to shear and crushing as the cement does after attaining its best strength. The field test necessary to investigate these points is a critical examination, with or without a microscope, and this may not necessarily be exhaustive. Tests supplementing the ocular examination are needed. Organic acids may take the shape of a colloidal film around the sand particles, preventing the adhesion of the cement. Washing is the remedy. While small amounts of some mineral clays or fine siliceous silts are known to increase the strength and densities, others commonly contain vegetable matter which is entirely injurious to the setting up of concrete.

In spite of the specified graduation up to a ¼-in. size, much sand is used which has no coarse particles at all, or else it has too great an excess of fine material. Tests on Lafayette bank sands for density in a lean 1:6 mixture as to strength, determined by 2-in. compression cubes, indicates that the densest mortar and the strongest are given by the straight-line gradation of sizes of particles. In the determination of the density on 1:3 mortars it was found that—the proportion of cement being constant—as a greater proportion of finer material is added to the coarser particles the density is practically constant until the straight-line gradation is reached. After that the addition of finer particles decreases the density. If a study of all fine aggregates indicates the straight-line gradation is best, the application commercially is simply to make up a sand in which 100 per cent passes the ¼-in. sieve and 50 per cent passes a No. 8 sieve.

Sloppy mixtures with standard Ottawa sand and good natural sands lose 25 to 30 per cent of their strength, while poor sands may lose 70 or 80 per cent. This loss is explained as being caused by the greater dissolving action of the water and, in connection with the abrasion of mixing, by the wearing down of the softer, poorer particles, turning them into mud, thereby causing a decrease in strength and setting properties of the concrete.

Street Mirrors for the regulation of traffic at corners of narrow and busy streets have been in use in Norwich, England, for 18 years. In the "Journal" of the Institution of Municipal and County Engineers Harold Collins, deputy city engineer, says that the mirrors are fixed about 7 ft. from the ground to a lamppost or standard at sharp intersections. Each mirror is about 4 ft. square. Mirrors slightly convex, to reflect a greater length of road, are sometimes used. Opinion is divided as to the utility of the mirrors.

Track Standards for New Rapid-Transit Lines in New York

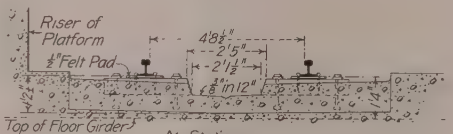
By ELLSWORTH L. MILLS

Engineer of Tracks, Subway System, with Gibbs & Hill

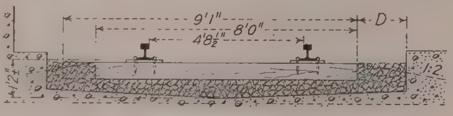
THE Public Service Commission, First District, New York, has undertaken for the first time to design the details of track work and buy the material necessary for track construction. The commission contemplates handling this material just as a railroad would, ordering it direct from the material contractors and installing it as the tunnel and elevated work now under way in New York progresses sufficiently to permit of so doing. It was not deemed advisable to let a general contract for the work, as the construction would have to be carried on sometimes under traffic.

Standardization and Mileage

In order to obtain a standard track construction throughout the dual system (comprising not only the city-built lines, but also the lines now owned and being built by the operating companies, all of which the city has the right ultimately to take over) the commission invited the New York Municipal Railway Corporation and the Interborough Rapid Transit Company to confer with its engineers. The result has been the adoption of a set of standards and specifications which will apply to all new rapid-transit lines whether built by the city or by the operating companies. The draft made for the public hearing of these specifications is now obtainable from the commission. This draft is subject to some minor modifications as noted therein.



At Stations



At Other Points

Types of Standard Roadbed

When distance D is greater than 18 in., or when wall between tracks is missing and distance between ties of adjacent track is greater than 2 ft., ballast is to be sloped off from end of tie as shown by dotted lines.

The accompanying table gives the approximate mileage of the dual system. The tabulation shows approximately 322 miles of new single track to be constructed according to the standard specifications.

The floor of the subway will be a concrete invert with a minimum distance of about 1 ft. from the base of the rail to the top of the concrete. The elevated structure will be the usual type of construction with girders on tangents spaced approximately 5 ft. on centers under the tracks.

The track will consist of 100-lb. A.R.A. type "B" open-hearth rail on tangents and curves with a radius of more than 700 ft. and manganese rail of the same section on curves with a radius of less than 700 ft. All curves under 2000-ft. radius will be guarded, using different flangeways for various degrees of curvature. The rails will be secured to 6 x 8-in. x 8-ft. untreated yellow-pine ties with 6-in. cut spikes, except at stations. The guard rails are to be fastened to the running rails at intervals of about 3 ft. with a 15/16-in. bolt passing through an adjustable filler block. Spiral spring nutlocks are to be used on these bolts. Malleable-iron rail braces will be installed on the guard rail and on the main outside rail on all curves under 700-ft. radius. Rolled open-hearth-steel, flat-bottom, shoulder tieplates will be placed on all ties and will be ½ in. thick by 7½ in. wide, with a minimum length of 9 in. Owing to the rather small distance between the floor of the subway and the bottom of the ties, trap-rock ballast is ordered, not too large to pass through a 1-in. and not too small to pass through a ½-in. ring.

Mileages of New York City Transit System

	Track miles
Interborough Rapid Transit Company	
Existing subway	73.0
Existing elevated lines	118.0
Subway and elevated lines for construction jointly by city and company	146.8
Elevated railroad extensions to be constructed by company	10.4
Third-tracking on elevated roads to be constructed by company	10.5
	358.7
New York Municipal Railway Corporation	
Existing elevated lines	105.0
Subway and elevated lines for construction jointly by city and company	110.4
Elevated extensions for construction by company	35.3
Third-tracking and reconstruction by company	9.3
	260.0

Station Track and Special Work

At the stations the rails are to be secured with screw spikes to 6 x 10-in. creosoted blocks embedded in concrete. To prevent the track from getting out of alignment, these blocks will be anchored to the foundation by two 15/16-in. bolts. The tieplates in this construction will be rolled with a ridge or boss for the support of the head of the screw spikes. Cow-hair felt pads ½ in. thick are called for between the ties and the tieplates. This type of track was adopted in order to eliminate the filth that accumulates at the stations.

The section of the subway is usually such that crossovers are very costly; therefore sharp turnouts are used extensively. To keep the radii as long as possible under these conditions and eliminate many of the ill effects of sharp curves, a special design for turnouts has been made to ease the curves as much as possible, obtaining in most cases an actual radius very near to the theoretical radius. Wherever the equipment will permit, the rail will be offset and the switchpoints protected by a cast manganese housing. Rolled manganese rail will be very extensively used throughout the special work.

Bids for the material will be asked for on a unit-price basis, and approximate estimates of the amounts required will be furnished the bidders. A sufficient quantity will be ordered to take care of the expected requirements of the city for an extended period. This material will be stored in a yard and requisitioned in a manner similar to that followed by railroads. This method of handling the work is expected to produce very efficient results and do away with a middleman's profit and the large amount of wastage of left-over material. All used material will be returned to the storeyard, and after all the track work is completed it will be turned over to the operating companies for their use in maintenance.

Japanese Road Development

Features of Locally Financed Triple System—National, Provincial and Village

PARTICULARS of the highway administration of Japan were recently obtained by the "New York Highway News," the bulletin of the State department, from H. Nakahara, minister of roads, bridges and canals of the eastern empire, who is studying American methods.

Road and bridge building are under the control of the Japanese department of home affairs. Highways are classed as national, provincial or village roads—"kokudu," "ken-do" or "ri-do." The national roads lead from Tokio to the great Ise shrine, to the headquarters of the army divisions and to all the naval stations and sites of prefectural offices, and are 42 ft. wide. Provincial roads, which connect prefectures, towns and important seaports, are from 24 to 30 ft. wide. The village roads have no standard width. There are 5243 miles of roads of the first class, 22,040 miles of the second and 231,078 miles of the third—a total of 258,361 miles. The fact that Japan is made up of about 4000 islands accounts for the fact that, besides a vast number of pontoons, wooden bridges and earth embankments, there are 119 structures of iron and 61,835 of stone in the system.

Cost of construction and maintenance falls chiefly upon the prefectures, cities, towns and villages, though the State treasury occasionally aids. The average yearly expenditure between 1896 and 1906 was about $4,890,000 for roads and $1,610,000 for bridges.

There are 15,000 miles of improved roads, gravel or concrete, which need resurfacing. The more important thoroughfares in Tokio are of waterbound macadam, but asphalt, concrete, wood block and granite are being experimented with. As the island abounds in suitable rock and gravel, the use of these metals is desirable.

Automobile travel is principally confined to the larger cities, and the motor truck has made absolutely no headway. It is realized that the rapid development of extensive motor traffic is to be looked for, and the Government in 1910 decided to make a fixed annual grant for road improvement.

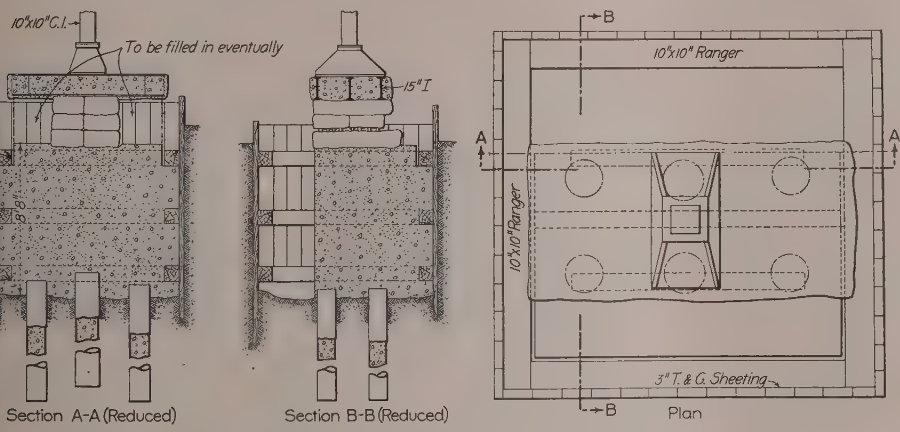
SERIOUS FIRES STARTED BY LOCOMOTIVES in Canada during the season of 1912 are reported to total 164, while 36 fires started on the railroad right-of-way are ascribed to tramps, campfires and unknown causes.

Underpinning Near Deep Subway Excavation

Special Connections of Columns and Piers to Needlebeams—Deep Concrete Piers and I-Beam Footings on Pipe Columns

SECTION 2A of the Lexington Avenue subway in New York is a four-track steel and concrete structure extending about 170 ft. on Canal Street and 530 ft. on Broadway, where its width is increased by the construction of station platforms outside the tracks, thus giving it a maximum width of 78 ft. over all and a maximum depth of about 46 ft. below the surface. The soil encountered consists chiefly of coarse and fine sand, with clay and some small boulders and a large amount of ground water, chiefly from local sources, which, when operations were commenced, extended to within 7 ft. of the surface of the ground and has since been lowered about 20 ft. by constant very heavy pumping. On Broadway the structure occupies nearly all of the 80-ft. width between house lines.

A few of the buildings on Broadway are steel-frame office structures with grillage foundations, but most of them and those on Canal Street are old brick buildings from four to six stories high, with stone wall and column foundations resting on the sand from 9 to 12 ft. below the surface of the street. In order to prevent undermining by the deep excavations close alongside, all were underpinned with new footings carried down below subgrade of the subway before the general excavation was made below their footings.



Column Underpinned with I-Beams on Permanent Footing in Sheeted Pit

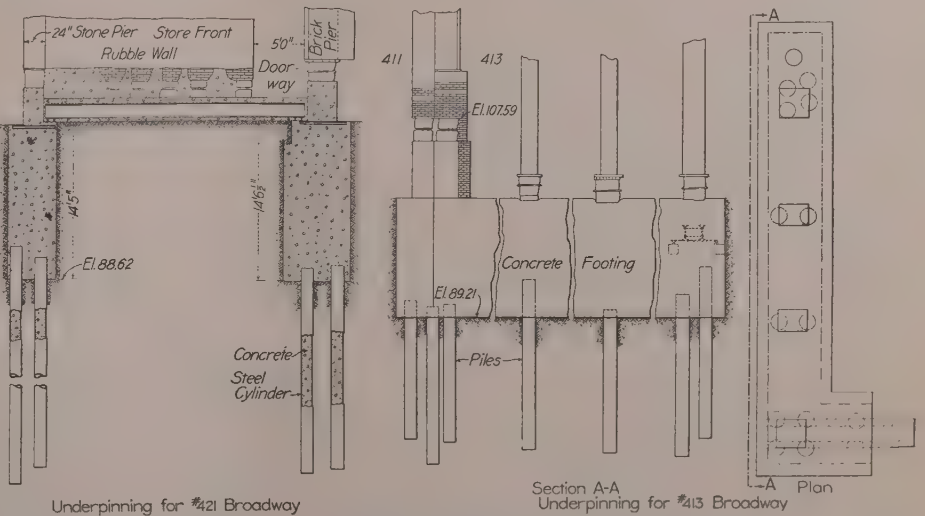
Generally this was effected by supporting the walls, columns and piers on needlebeams consisting of long and heavy I-beams with jackscrew adjustment to compensate for any settlement under their support, while sheeted pits were excavated to a sufficient depth to permit the driving in them of sectional steel pipes 12 and 15 in. in diameter. These were driven to a depth of 15 to 30 ft. as required, cleaned out and filled with concrete. New footings were then built on them, the weight of the building was transferred to them and the needlebeams were removed. Some of the standard and special features of this work are here described.

SHORING AN IRON COLUMN AND BRICK PIER

In the front of the five-story building at 425 and 427 Broadway a 24-in. rectangular cast-iron column in one building is in contact with a 24 x 30-in. brick pier in the adjacent building, and as neither pier nor column had any lateral extensions suitable to afford bearing for the shoring, and it was not practicable to support them from under their bases, special connections were provided for transferring their weight to the needlebeams. A little above the cellar floor twelve holes were drilled in each of two opposite faces of the column to receive 1¼-in. steel tap bolts screwed through the webs of two short horizontal channels, which were thus securely connected to the column to form wings or shoulders, with their bottom flanges seated on the top flanges of a set of I-beam needles on each side of the column.

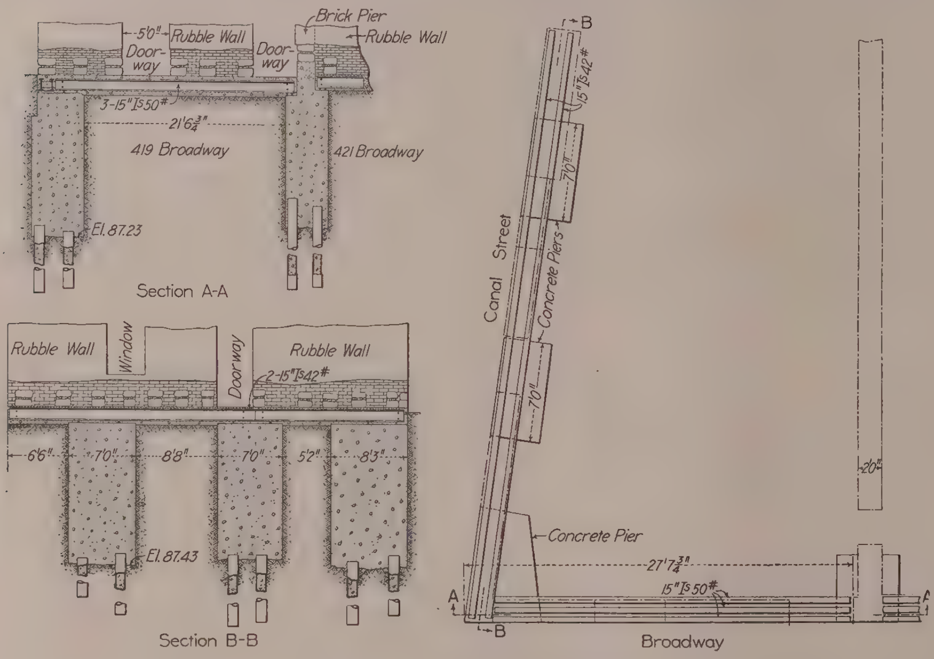
Horizontal slots parallel to the needlebeams were cut in opposite faces of the brick pier just below the top flanges of the needlebeams. One of the needlebeams was set in the outer slot and its top flange wedged to bearing against the brickwork, and in the other slot, between the pier and the cast-iron column, short 12-in. I-beams parallel to the needles were placed, with their top flanges bearing against the brickwork. The 12-in. I-beams were suspended at both ends from short saddlebeams on the flanges of the needles and the dead work of the brick pier was removed below the I-beams.

The needlebeams were supported on



Wall Supported on Three 15-Inch I-Beam Girders and Columns Supported on Concrete Retaining Wall Footing jackscrews on cribs or sills far enough apart to provide clearance between them in which a sheeted pit was excavated about 18 ft. deep, affording clearance for six 12-in. pipes to be driven under the pier and column. The pipes were driven by a Goubert hammer operated by compressed air. They were driven in sections 6 to 8 ft. long and connected by inside cast-steel sleeve couplings. As they were driven the contents were excavated by a small canvas bag with steel cutting edge and by a very small Hayward orange-peel bucket.

After the driving was completed a form was built around the upper ends of the pipes and filled with 1:3:5 Pennsylvania brand Portland cement concrete, making a pier about 18 ft. high, to which the support of the column will eventually be transferred through wedging stones in the usual manner. The rate of sinking the pipes varied greatly with the conditions encountered and was sometimes very much retarded by small boulders. Part of these was removed directly by the orange-peel bucket, but sometimes it was necessary to break them with bars first.



Old Footings Underpinned with Girders on Concrete Piers and Piles

Underpinning a Grillage Foundation

The center column in the front of the building at 416 Broadway has a 10 x 10-in. rectangular section and carries an estimated load of 200 tons. It was seated on a grillage of three 15-in. I-beams without bolts or separators, embedded in the concrete footing. After the weight of the column was transferred to needlebeams a 10 x 10-ft. pit was sheeted down under it to a depth of about 11 ft., six 15-in. pipes about 25 ft. long were driven into it, and a concrete pier was built over them up to the building line, leaving a clear working space of about 3 ft. beyond it. Three two-course sets of granite stones were placed on top of the pier and folding steel wedges were driven between them to give bearing under the center of the old grillage. The full load of the column was taken in this way and will be maintained on the new foundation until there is no further probability of settlement, when the spaces both sides of the wedging stones will be built up solid, giving continuous support to the grillage.

The four-story brick building at 419 Broadway is on the corner of Canal Street and has a total frontage of about 70 ft. on the subway excavation. There are no wall columns, except three on Broadway, the floorbeams being carried directly on the brickwork, with foundations originally seated on the sand about 10 ft. below the surface of the street. These walls were shored on long steel needles while excavations were made under them and the old footings removed, and four new concrete piers with pipe foundations were built under them in the manner previously described. Sets of two or three 15-in. I-beams, bolted together with separators to give them a width equal to that of the footing, were seated on the piers and inclosed in solid concrete, protecting and stiffening the wall girders, to which the weight of the building was transferred through sets of wedging stones and brick underpinning.

Underpinning Columns on a Continuous Pier Footing

The six-story building at 413 Broadway has four cast-iron front columns, which, together with the brick pier at the front corner of the adjacent building, have been underpinned and the load transferred in the usual manner to a long concrete pier 5 ft. thick and 8 ft. deep which is continuous across the front of the building and is supported on fifteen 12-in. pipe columns extending 15 ft. below the bottom of the pier. The pier concrete entirely fills the pits in which the pipes were sunk, permanently inclosing the waling pieces and sheeting, and is extended at one end 3 ft. beyond the main body of the pier at right angles to it, having an L-shape footing for the support of an exterior column about 8 ft. on centers from the corner column. A pair of 12-in. I-beams were set on top of the four pipes under the corner column so that the weight of the column above serves as an anchorage, and the I-beams are supported on a pair of fulcrum pipes in the extended footing and project 4 or 5 ft. beyond them to form a cantilever supporting the exterior column with an estimated load of 50 tons.

The subway construction is being executed under the direction of the Public Service Commission, of which Alfred Craven is chief engineer, Robert Ridgway, engineer of subway construction, and T. O. Shipman, division engineer. The underpinning was executed by the O'Rourke Engineering Construction Company, general contractor for Section 2A. John Kirgan is superintendent and John J. Dooley engineer for the contractor.

Fireproofing of Wood

Abstract of Preliminary Report by Robert E. Prince on Investigation Carried On at Forest Products Laboratory

THE question of fireproofing wood was included in the investigations of the Forest Products Laboratory at Madison, Wis., during the early part of 1912. Results to date were presented by Robert E. Prince, of the laboratory, at the recent convention of the American Wood Preservers' Association. These notes were taken from his paper:

Very good results were obtained on commercially treated wood. The main part of the problem was not to find a method of fireproofing wood but to determine the cheapest and most efficient method, or, more definitely, to determine the relative value and efficiency of various chemical compounds in rendering wood non-inflammable or slow-burning.

The combustion of wood may be divided into two parts: (1) The driving off of the volatile gases by heat, and their ignition causing flame, and (2) the combustion of the non-volatile portions, analagous to charcoal, causing glow or incandescence, and the production of carbon monoxide, which is a combustible gas. It will readily be seen that the first part of the combustion is the most dangerous. The chief problem, therefore, is to render the volatile gases non-inflammable.

Most Efficient Chemicals

It has been found that the chemicals most efficient in rendering wood non-inflammable are those that sublime or decompose upon being subjected to intense heat. Non-combustible gases are given off in both cases, which, when mixed with the inflammable gases arising from heated wood, render them non-inflammable.

The chemicals selected for the first experiments were divided into the following two classes:

(1) Chemicals that "sublime" when subjected to intense heat whose vapors are not inflammable, such as ammonium sulphate and ammonium chloride; and (2) chemicals that decompose when subjected to intense heat, giving off non-inflammable gases, such as ammonium phosphate, sodium bicarbonate and oxalic acid.

A method used in previous fireproofing work has been to inject chemicals containing large amounts of water of crystallization, the theory being that the water is converted into steam, which creates a non-inflammable atmosphere without the wood. Under this method borax and alum were used. Borax also has the advantage of being a fusible salt, thereby protecting the wood fiber with a glossy coating. The manufacture of fire-retarding paints was also included in the tests.

Although this report is of a preliminary nature, results obtained to date are as follows:

1. Ammonium salts are of considerable value in fireproofing wood. It was impossible to ignite wood, under the test conditions, that had been treated with these salts.

2. Borax is of considerable value in fireproofing wood. It has not the value of the ammonium salts, but promises a means of lessening the cost of treating by using it with another salt of greater value.

3. From the good results already obtained it appears possible to devise a reasonably inexpensive method of rendering wood fire-retarding.

4. Of eight untreated species of wood tested, tamarack is the most fire-resistant. Redwood comes next, even though Western larch and noble fir were slower to ignite.

Hauling on Stiff Grade with an Oil Tractor at La Crosse

INVOLVED in the construction of the 5,000,000-gal. reinforced-concrete distributing reservoir at La Crosse, Wis., were 5500 tons of material hauled up a 10-per cent grade, 1600 ft. long, by a 25-hp Rumley oil-tractor trailing two bottom-dump, 2-yd. wagons at an average cost of 8½ cents per ton for oil and 2 1/3 cents for repairs. For the first two weeks teams were tried out, when the costs ran to 50 cents per ton. The bulk of the material consisted of concrete aggregate. A quarry exists in the hill on which the reservoir is located. The crusher is stationed below the quarry and loading bins are built below the crusher at a point so hemmed in by switch tracks that the wagons had to be hauled by team from the bins to the foot of the incline, a distance of about 400 ft. The same narrow quarters existed at the top of the hill, where a team was required to haul the load about 400 ft. to a dumping bridge. These two teams—one at the top and the other at the bottom of the incline—cost $9 per day. On the average 100 tons were hauled in a ten-hour day. Invariably the teams were able to have wagons waiting for the tractor, which made the round trip in twenty-five minutes. The maximum number of trips per day was twenty-eight and the heaviest load 5.7 tons. When hauling began a crushed rock surface county road existed, but this was pretty well worn out during the season.

Alvord & Burdick were consulting engineers for the La Crosse improvements, which comprise, in addition to the service reservoir, a change from the Mississippi River to twenty driven wells, 115 to 160 ft. deep, a new pumping station, and a 1,000,000-gal. receiving reservoir. The Turner Construction Company, of Des Moines, Iowa, was the contractor.

Wittenbergplatz-Dahlem Extension of Berlin Subways

Section through High-Grade Residential Territory Is Notable for the Artistic Construction of Stations and Stairways

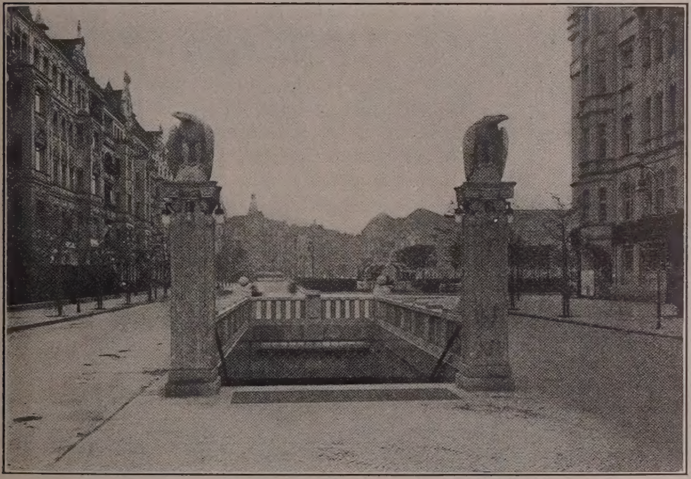
THE Berlin Elevated & Underground Railway recently opened its extension from Wittenbergplatz in Berlin to the fashionable suburb of Dahlem. The new line is in high-grade residential territory, and no expense was spared in the design and construction of the stations and station entrances. All of the buildings are very ornate, with sculptured granite or sandstone, metal portals and elaborate electric fixtures.

The entrance to the Wittenbergplatz station in Charlottenburg at the junction with the main line is prominently set off by its location in a small park or gore and additional perspective is furnished by the very wide streets around it. Directly behind the building are the sodded tracks of the Grosse Berliner Strassenbahn. The interior of the same station is finished in majolica tiling. Two fully glazed ticket offices are provided. The tiling near one ticket office carries a very large map of Berlin, while that on the side wall carries a plan of the station platforms with arrows to indicate the western and eastern sides. A post in the center of the station carries directions to the nearest adjoining streets and also a sign which presents the names of the stops in each direction.

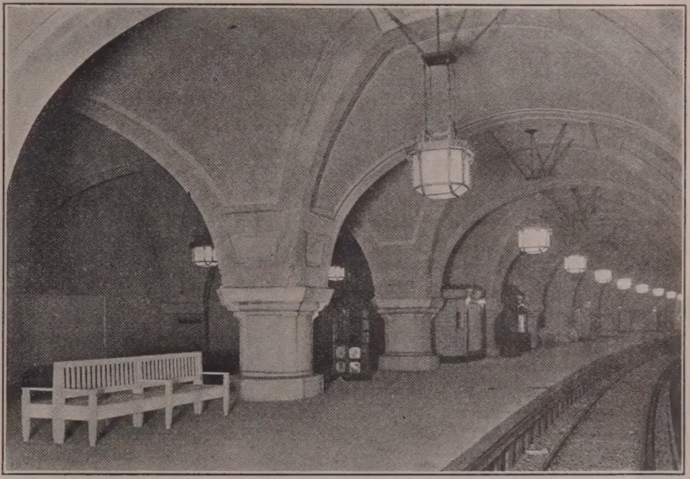
INDIVIDUALITY OF STATIONS

No two stations are alike. The same diversity in design is common to the interiors of the stations, all of which are finished in a different architectural and color scheme. Thus, there is a strong contrast between the plain doric columns and flat roof of the Breitenbachplatz station and the massive vaulted design at Heidelbergerplatz. The lighting fixtures also differ.

To permit the eventual operation of eight-car trains, all stations on the new line have island platforms 360 ft. long. At four stations double-loop tracks are provided for through and shuttle-service trains. The first train from the Dahlem terminal at Fehrbellinerplatz to Berlin leaves at 5:15 a. m. and the last train in is due at 1:30 a. m. The shuttle service begins at 6 a. m. and ends at 1 a. m. Within the Wilmersdorf territory the service is two and one-half minutes, in the intermediate territory toward Dahlem six minutes, and to the end of the line twelve minutes. The total length of the line, embracing nine stations, is 6.2 miles. All of the work was carried out by the Siemens & Halske Company, of Berlin, except that the tunnels and cuts at Dahlem were made by Haberman & Guckes, of Kiel.



Portal Carrying the Eagles of the Imperial Family



Vaulted Interior of Station at Heidelbergerplatz

Meteorological Aspect of Smoke Problem

FROM the meteorological aspect Pittsburgh's smoke problem has been investigated by Dr. Herbert H. Kimball, professor of meteorology, U. S. Weather Bureau. In one of the bulletins of the Mellon Institute of Industrial Research and School of Specific Industries he presents the following conclusions:

In the business section of Pittsburgh the limit of visibility is about one-tenth the limit in the open country.

On days when the presence of smoke or fog is recorded the chemical action of daylight is only 75 per cent of what it is on days when no smoke or fog is recorded, other conditions being equal.

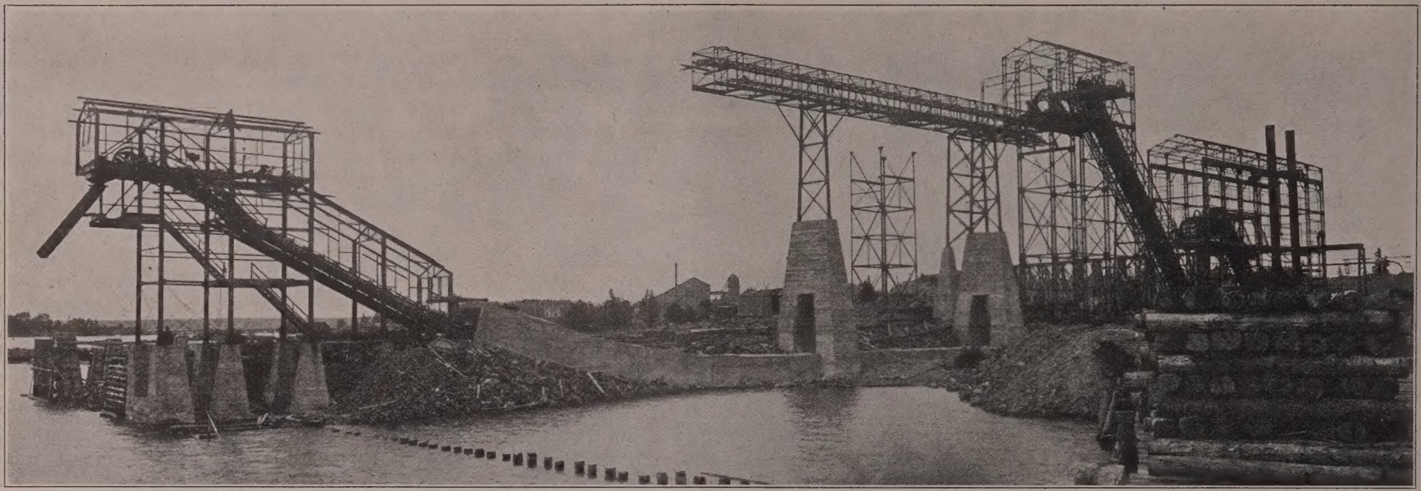
On smoky or foggy days with equal hours of sunshine the chemical action of daylight is closely related to the limit of visibility.

The chemical action of daylight in Pittsburgh is about 60 per cent of what it is in Sewickley, a small residential town.

There is evidence that in general the chemical intensity of sunlight in large cities is 25 per cent less than it is in small towns.

The effect of the smoke cloud upon the temperatures in cities is to materially raise the minimum temperatures, especially during the warm months of the year, thereby decreasing the diurnal range in temperature.

ZINC ORES are now being shipped from Australia to Norway for smelting in electric furnaces, and at present a 200,000-hp plant is being built in Tasmania for the smelting of the complex ores found in that region.



View of Partly Completed Plant, Loading Conveyor on Left and One of Concrete Tunnels in Center

Trap-Rock Crushing Plant at Bruce Mines, Ontario

Material Undergoes Three Size-Reducing Operations and Is Loaded on Vessels from Tunnels under Storage Piles

TO prepare immense bodies of trap rock at Bruce Mines, Ontario, 40 miles east Sault Ste. Marie, for use in road construction a large crushing plant has been installed at that point by the Martins International Trap Rock Company. The plant has a capacity of more than 500 cu. yd. per hour, and is laid out with a view to future enlargement.

Two 110-ton steam shovels, two large electrically operated churn drills and twenty-five 8-yd. steel automatic-dump quarry cars constitute the quarry equipment. The material is hauled from the quarry and dumped direct into a Superior jaw crusher with a 5 x 7-ft. receiving opening. Reduced by this crusher, which is the largest built, to about 10-ft. size, the material passes over a short grizzly, the fines falling on a 4 x 76-ft. pan conveyor and the oversize continuing to two No. 9 McCully crushers. The material from the McCully crushers is discharged to the same pan conveyor, which elevates the entire output and delivers it to a 7 x 18-ft. McCully stone screen. From this screen the fines are discharged upon a 4 x 116-ft. pan conveyor, the oversize going to two sets of 54 x 24-in. Superior rolls. The rolls in turn discharge into the same conveyor, which carries its load to a second 7 x 18-ft. screen. From this screen the larger sizes are delivered to storage piles by gravity and belt conveyors, the fines to a cross-conveyor which takes them to two 4 x 12-ft. double-deck screens, whence other conveyors take them to other storage piles. From the storage piles the material is loaded on vessels by belt conveyors in tunnels under the piles.

Storage is provided for approximately 80,000 cu. yd. There are two rows of storage piles, and under each row is a concrete tunnel with feeding hoppers in the roof. Each tunnel is provided with a 40-in. belt conveyor 330 ft. long, capable of loading stone at a rate of 750 tons per hour. Near the discharge end of each conveyor is installed a Merrick weightometer, which automatically weighs the material. With the system the largest vessels can be loaded within a few hours.

The layout is such that a duplicate plant can be installed at a later date, or the storage of the present plant can be increased to 120,000 cu. yd. by the addition of one concrete tunnel with loading conveyor and overhead distributing conveyors. The crushing machinery was furnished by the Power & Mining Machinery Company, of Cudahy, Wis.

Bridge Foundation on Gravel

Abstract of Paper by John W. Mueller, Newcastle, Indiana, Read at Meeting of Indiana Engineering Society, Jan. 23

FAILURES of bridge foundations in the spring flood last year in most cases were confined to gravel channels and were due to high velocities. The duration of high velocities during high floods will, to a great extent, govern the effective depth of erosion. As soon as the material around the lower region of the masonry or piling is reached a displacement begins and security is lost, and often in a few minutes stability also.

One method of treating a stream bed of gravel for a small river is to lay a concrete pavement beyond the end of the foundations on both up and down stream sides of the bridge, not less than a distance equal to three-quarters of the high-water head taken at the deepest part of the channel. The upstream edge should be extended down into the stream bed to form a cutoff wall. The slab should be not less than 9 in. in thickness and reinforced.

In large channels foundations must be sunk below any chance of erosion. General practice in Indiana has been to carry the excavation down 8 to 12 ft. below stream bed and then drive wood piling under a specification calling for a sinking to refusal by a 2500-lb. hammer falling 25 ft. To make the piles stable against scour they should be driven at least 20 ft. below the bottom of the excavation, which should be from 12 to 15 ft. below the channel bed. A water jet will be required in sand to effect this depth of penetration.

HIGHWAY FORESTATION, as practised by the Massachusetts Highway Commission, resulted in the planting of 16,242 trees along Massachusetts highways in 65 towns during the nine years prior to Jan. 1, 1913. In 1912 more than 1000 trees were planted, in addition to 781 that had to be replaced. Quick-growing trees and hedges are used in place of guard rails, affording economy as well as improved appearance along roads.

Book Reviews

The New York State College of Forestry at Syracuse University, in co-operation with the U. S. Forest Service, Department of Agriculture, has issued a bulletin dealing with the wood-using industries in New York State. It contains 213 pages, 7 illustrations and 55 tables. It gives a summary table of quantities and percentage of lumber used annually for each of forty-seven industries, the unit and total cost, and the proportions grown in and out of New York. Short sections devoted to each of these industries follow, accompanied by tables classifying the information in the summary table according to different kinds of wood. Appendices, which comprise the last 105 pages, include a table of the uses to which different species of wood are put, and a directory of the users. It may be of interest to know that the Forest Service is preparing bulletins concerning the wood-using industries in various other States.

SCIENTIFIC AND TECHNICAL SPANISH for Colleges and Technological Schools, with Vocabulary and Notes. By Cornelis De Witt Willcox, Lieutenant-Colonel, U. S. Army, Professor U. S. Military Academy. Cloth, 5 x 7½ in.; 588 pages. New York, Sturgis & Walton Company. $1.75 net.

This book, written in very simple Spanish, is intended mainly for engineers who propose practising their profession in the Spanish-speaking Americas. The author has, therefore, chosen his subjects from among the technical sciences. The work treats of the following subjects: Physics, chemistry, the storage of modern explosives, electricity, water powers, steam, compressed air, turbines, transmission of power, minerals, bridges, railroads, surveying, topography, geography, the automobile, aeronautics, submarines and the battle of Santiago de Cuba. The work contains a 61-page technical vocabulary.

The report of the Cherry Creek Flood Commission of the flood at Denver, Col., in July, 1912, which was submitted on May 20, 1913, has appeared in printed form. The report, as submitted, cannot be considered complete, since the investigations and surveys were not made in as great detail as was desirable. The report comprises 159 pages and begins with a résumé of previous floods as far back as 1864. The next chapter treats the flood of July, 1912, the precipitation causing it, the flood discharge as observed and calculated (actual maximum discharge considered as having been larger than that indicated by the gaging station records) and the damage resulting from the flood. The plans proposed for controlling eventual future floods are dwelt upon in some detail. The main part of the report ends with a summary, conclusions and recommendations. It is signed by A. Lincoln Fellows, J. B. Hunter and C. A. Trease. There is an appendix containing four previous reports submitted by the same commission and reprints of newspaper clippings of the floods of 1864, 1875, 1876, 1878 and 1912. The appendix ends with a report on methods of flood relief adopted in foreign countries; this report is reprinted from the 1911 report of the Flood Commission of Pittsburgh.

THEORY OF IRREDUCIBLE CASES OF EQUATIONS. Part II. By Charles Edgar White, Professor of Mathematics, West Virginia Wesleyan College. Cloth, 6¼ x 9¼; 90 pages; 16 illustrations. Published by the Author, Buckhannon, W. Va.

In Chapter 1 are shown the methods of deriving trigonometric formulæ for the solution of equations of the third degree. The solutions of irreducible cases of equations of the third, fourth, fifth, sixth, seventh and higher degrees, the methods of computing trigonometric functions and their logarithms, a method of computing logarithms, the methods of computing the lengths of chords, subchords and middle ordinates, and the method of finding the length of an arc are dealt with in Chapter 2. In Chapter 3 the practical trisection of an angle and the trisection by integraph, ruler and compass are described. Chapter 4 deals with methods of deriving the equations upon which the inscription of polygons depends. Chapter 5, the last one, gives the practical construction with rule and compass of regular polygons of seven, nine, thirteen and seventeen sides and multiples thereof.

The book is well written and will afford interesting reading to engineers and others interested in a scientific treatment of mathematical problems involving irreducible equations.

A TREATISE ON ROADS AND PAVEMENTS. By Ira O. Baker, Professor of Civil Engineering, University of Illinois. Cloth; 6 x 9 in.; 698 pages; 171 illustrations; 68 tables of dimensions, cost, etc. New York, John Wiley & Sons, Inc. $5 net.

While there are numerous books on highway and pavement construction, there is still room for an up-to-date exposition of this subject given in a clear, comprehensive, yet concise style. Probably no text on roads and pavements has met the needs of the student of highway engineering better than Professor Baker's "Roads and Pavements." The author's clear, concise, logical presentation has made it a delight alike to the instructor, the student and the practising engineer. It has become, however, obsolete in some matters and was entirely lacking in an exposition of the advance in road treatment required by the sudden and great development in motor traffic.

A part of this deficiency has been supplied by a chapter on automobile roads and concrete pavements. This chapter is written in the same clear, concise, analytical method which the author has so happily used hitherto in all of his text work.

The effect of motor traffic is explained. In 42 pages of additional text are treated clearly first, the suppressing of dust by sprinkling, the use of deliquescent salts, oils, proprietary and emulsion compounds and tars; second, the forming of a protection coating over old gravel and macadam roads and old pavements by means of oil, paint coats of tar and asphalt and the application of bituminous carpets; third, the construction of bituminous macadam, and asphaltic concrete both by the penetration and mixing methods; and fourth, the construction of concrete pavements by five different processes, laying most emphasis on the single and double course methods adopted in Wayne County, Michigan, and closing with a very good exposition of the expansion problem in concrete roads and how to deal with it.

The original text was not altered except to add a few pages of critical analysis to the author's already rather too lengthy criticism of Circular 19 and Bulletin 49 of the Road Inquiry Office of the U. S. Department of Agriculture. That the criticism was needed is admitted, but the author has used more space than he usually gives to destructive policies.

It is not well to close this review without a friendly criticism of the author in that he has allowed this opportunity to pass without revising his chapters upon concrete foundations, particularly regarding machine mixing and wet or dry concrete, and upon the brick-rattler test, with perhaps some revision and somewhat fuller discussion on asphalt and bitumens.

Letters to the Editor

Mixed vs. Penetration Roads

SIR: The very sound editorial in your issue of Jan. 24, page 89, under the above heading suggests making prominent some conclusions derived from the experience of the writer extending over six years in the construction and behavior under travel of the two types of road.

The merit of surfaces constructed by these methods may be considered from two points of view—the relative ease and uniformity of construction and control thereof and the relative durability under travel. Through lack of skill in doing the work, through carelessness, together with insufficient inspection and control, poor road surfaces can be constructed by either method, and it is possible that carelessness in construction can be more readily concealed in the penetration than in the mixing process, but service tests have shown that where equal care is used roads constructed by the penetration process are superior or equal to the other form of construction, if the broken stone is of satisfactory character and size.

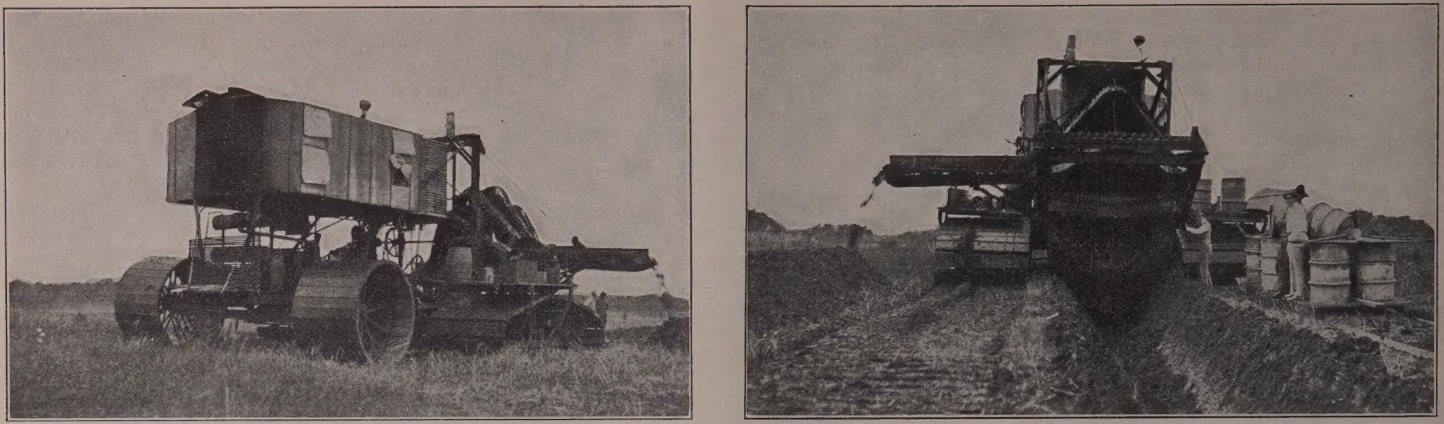
There is no limit to the size of the particles of broken stone that may be used in a penetration surface up to 2½ in. Stone of this description, if of uniform size to the exclusion of smaller particles, is a more stable material than if of smaller size. This has been shown by experiments conducted by the Pennsylvania Railroad in maintaining track ballast of various sized stone. The cost of maintenance of the larger stone was much less. The reason is not far to seek for using large stone of one size in road surfaces in preference to smaller or graded material. Each piece of stone then has a bearing on others of the same size and can be well locked together. If smaller size particles are introduced into the surface the result is to wedge the larger stone apart and prevent complete interlocking and the stability which is obtained with one-sized stone.

A similar result can be obtained by the mixing process with the same size stone, if it is mixed on a mortar board, by hand, with the bitumen, or in one of the revolving forms of concrete mixers without the aid of heat, the stone being previously dry or warmed to a proper temperature outside the mixer; but it is impossible to use stone of this description in any mechanical apparatus provided with blades, as these are readily broken off by stone as large as 2½ in. in diameter. Such a road was constructed by the mixing process in 1909 on Western Avenue, Gloucester, Mass., which is to-day in excellent condition. The mixing was done by hand. In the same year the Hudson-Hollowville road, County Highway 650, was constructed in New York State by the penetration process in an excellent manner, using 1½ gal. of bitumen to the square yard in penetrating the stone and ½ gal. for the seal coat. This road is to-day the equal of the Massachusetts surface, constructed in the same year, but most of these road surfaces which are built by the mixing method require, because of the mixers in use, much smaller stone and are not as stable. They have not worn, for this reason as well as for others, nearly so well.

In constructing the road with a mixed surface, especially with warm stone, the amount of bitumen that can be applied to it is no greater than that which will adhere to the surface of the stone. An excess would be lost in putting the stone in place after leaving the mixer. For this reason the amount of bitumen in the road surface is limited. On the other hand, in building a surface by the penetration method a limited excess of bitumen can be used, and this will exist in the road surface as a reservoir of material to constantly enrich the road under travel, where it would dry out and ravel had the mixing method been employed. This is particularly the case where the oil asphalts are in use, as such material melts to a very thin liquid and consequently does not adhere to the broken stone to the amount that would be desirable.

Where smaller stone, 1 in. in diameter or less, is employed in the mixing process, especially with the addition of finer material, great instability is observed, for reasons which have been already given. If a graded stone is employed, the voids therein must be filled, owing to its instability, with a bituminous mortar in order to produce a stable bituminous concrete.

In the light of these observations and the conclusions to be derived from service tests, it seems to the writer that the penetration process, although it requires careful work and close supervision, will, if these are observed, lead to results which are superior to work done by the mixing method. It must, of course, be remembered that a suitable amount of bitumen must be used in the penetration process, and that this cannot be reduced below 2 gal. to the yard, inclusive of the penetration and seal coat, for 3-in. work. There is much work in New York State where the quantity has been reduced to 1½ gal. where the results are not satisfactory, nor would they be with the same



Cutting Trench in Florida Everglades with Ditching Machine Containing Quarters for Eight-Man Crew

amount of material introduced into the road by the mixing process. With the desire to prevent bleeding with the asphalts of lower melting point and which are less stable under the summer's sun, the amount of bitumen has, in many instances, been reduced below the point of safety, with results which are easily distinguishable after one or two years' continuous travel. It seems to be safe to say that many engineers of the keenest powers of observation and the most extended experience are coming to the conclusion that the penetration process, in skilled hands, is to be preferred to the mixing method.

Attention should be drawn to one other point in connection with the roads of this type which should be considered in order that an adequate return may be obtained for the cost of construction. The surfaces should not be allowed to deteriorate to such an extent that they will require maintenance, due to any defects therein. An experienced observer will readily detect the time when the surface presents a hard and dry appearance, which is the forerunner of raveling. When this is recognized a carpet or seal coat or suitable asphaltic oil should be applied, and this should be regarded as a preventive treatment as distinguished from the correction of defects or deteriorations in the road surface. It is evident that sufficient consideration is not given to this aspect of the case, but the result is well illustrated with numerous roads on Long Island where this preventive treatment has been successfully applied.

Clifford Richardson,
New York. Consulting Engineer.

Sir: I have read with much approval your editorial "Mixed vs. Penetration Roads" in the issue of Jan. 24 and heartily agree with your statement that "it is useless to tell public authorities and engineers that penetration construction should be abandoned."

This city has built since 1908 about 60,000 sq. yd. of bituminous macadam pavement, the greater portion of which has been constructed by the penetration method and the remainder by the mixing method. All the work was done by day labor under the same supervision and by practically the same men and equipment using the same materials. We have found the penetration method to be much more satisfactory in every way and to be about 33 per cent cheaper.

We have found in the mixing method that if we use a binder "hard enough" to stand high summer temperatures we must heat our aggregate. If we get stone too hot the binder does not adhere in sufficient quantity; if too cold, the mixture is hard to spread evenly and impossible to compact and finish properly. If we use a binder that is softer it is difficult to finish to an even surface, and in hot weather the pavement crawls or works into ridges under traffic and bleeds where there is an excess of bitumen. We find that a penetration pavement properly constructed does not bleed in hot weather, does not crawl under traffic, and that it can be finished smooth and kept smooth by proper treatment for an indefinite period.

The requirements for such a pavement are a properly prepared subgrade, a hard, tough stone aggregate, such as the trap rocks, a binder that will not be too brittle in cold weather nor too soft in warm weather, a proper distributor for applying bitumen, a 10-ton roller, and care and good judgment in using materials. All of the above, except the distributor, are required in the mixing method, and much in addition, such as heaters, mixers, etc.

In the penetration method the stone aggregate is placed and rolled until the larger voids are filled and the particles of stone have come to a more or less firm bearing. The smaller voids are then filled with the bitumen and the particles of stone stick together so that they are not readily moved.

In the mixing method the particles of stone may or may not come to a bearing against each other. If they do you have good results; if they do not, you have a pavement which crawls and works into pot holes. It is an exceedingly difficult problem to mix a sticky bitumen and broken stone of the size used in such a manner that there will be sufficient particles of stone, properly bearing on each other, in each cubic foot of pavement. Eternal vigilance and painstaking care are required for a good bituminous pavement of any description; but it has been my experience that no more, if as much, is required when the penetration method is used.

Charles A. French,
Laconia, N. H. City Engineer.

Costs of Ditching in the Everglades

Sir: In the Current News section of your issue of Jan. 3, 1914, you published an article on a ditching machine. I inclose herewith two views of a ditcher which is now operating on the property of the Everglades Sugar & Land Company in Dade County, Florida. This machine was built by the Buckeye Traction Ditcher Company, of Findlay, Ohio, but was completely remodeled by our forces to meet the conditions as found in the Everglades. The machine weighs 37 tons and is equipped with a 45-hp gasolene engine. The bearing on the ground is approximately 350 lb. per square foot.

Living quarters are provided for the crew on top of the machine, as shown in the photographs. This house will accommodate eight men. An independent electric generator furnishes light for the living quarters and for a searchlight, which makes it possible to run at night. The machine cuts a ditch 9 ft. wide on top, 2½ ft. wide at the bottom and 5 ft. deep at an average rate in sand and muck of 8 ft. per minute, or 480 cu. yd. per hour. It has cut 1 mile of ditch in ten hours, or 528 cu. yd. per hour. Our records for December, 1913, show a total of 43,630 cu. yd. of material excavated at a cost of $0.0287 per cubic yard, including overhead expense, fixed charges on the machine and cost of clearing line. Some exceptionally hard sand cutting and heavy clearing were encountered during the month.

From Jan. 1 to Jan. 23 the machine has excavated 58,630 cu. yd. of sand and muck at a total cost of $0.0240 per cubic yard. Owing to the fact that the muck soil is too soft and spongy to permit of transportation by animals, the machine must carry supplies for an 8-mile run, 4 miles out from the canal and back, cutting in both directions on lines ¼ mile apart.

W. J. Kackley,
The Everglades Sugar & Land Company.
Miami, Fla.

Railway Casualties in Canada for the fiscal year 1912-13 have just been announced in the report of the Canadian Railway Commission tabled at Ottawa on Jan. 23. According to these figures 643 persons were killed and 2315 were injured during the twelve-month period covered. Of the total number killed, 39 per cent were trespassers on railway property, while only 3.26 per cent of those fatally injured were passengers. More than three-fourths of the fatalities among passengers were due to what are considered "preventable causes on the part of the passengers themselves," so that the actual number of passengers killed in what might be termed causes preventable by the railway company was a little less than one per cent. The railway employees killed constituted 47 per cent of the total. Passengers constituted 17.9 per cent of those injured, and 18.3 per cent of the injured passengers were hurt by causes which they themselves could have prevented. Employees received 71.8 per cent of the total number of injuries.